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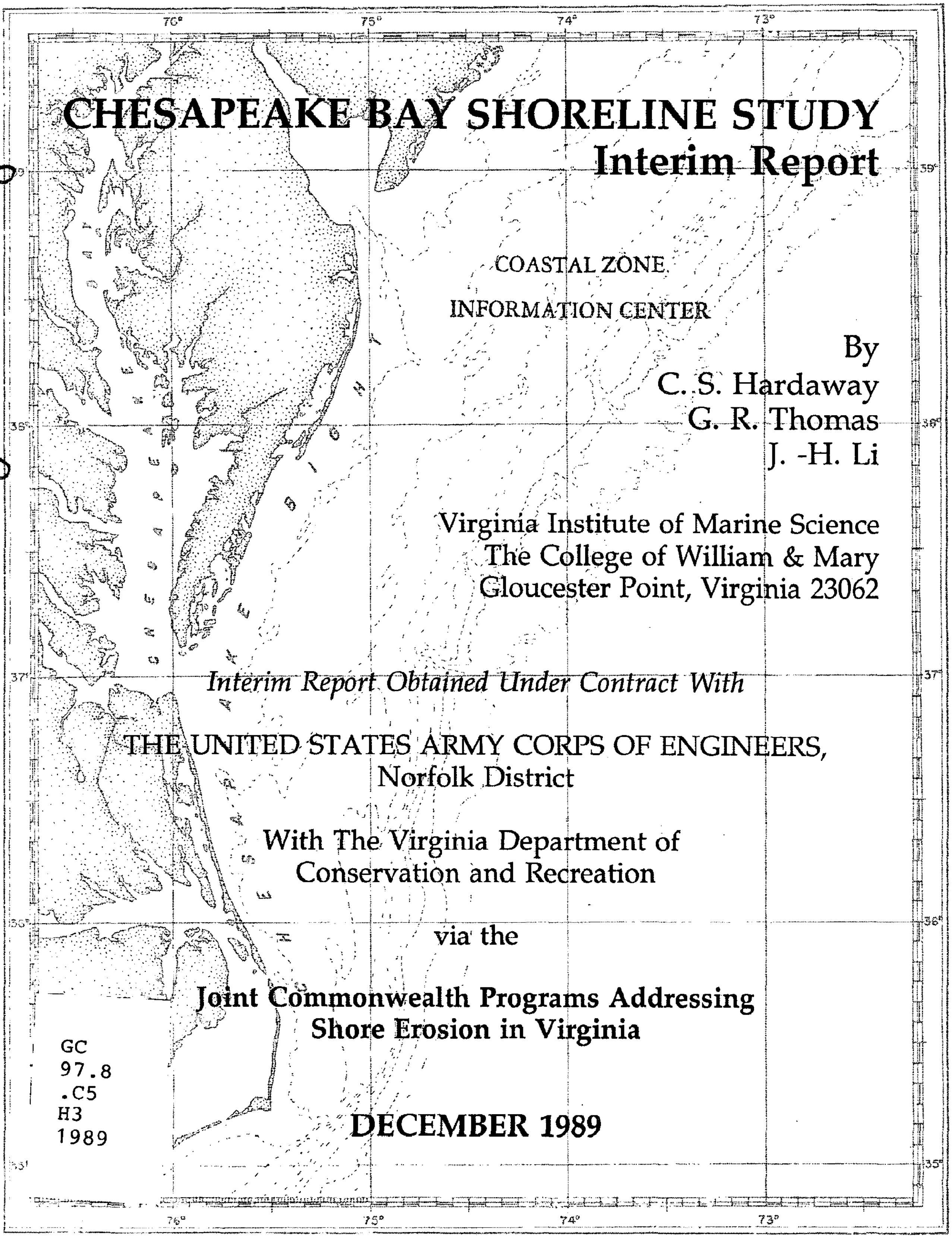

CHESAPEAKE BAY SHORELINE STUDY
Interim Report

By
C. S. Hardaway
G. R. Thomas
J. -H. Li

Virginia Institute of Marine Science
The College of William & Mary
Gloucester Point, Virginia 23062

Interim Report Obtained Under Contract With

THE UNITED STATES ARMY CORPS OF ENGINEERS,
Norfolk District

With The Virginia Department of
Conservation and Recreation

via the

**Joint Commonwealth Programs Addressing
Shore Erosion in Virginia**

DECEMBER 1989

Virginia · Coastal Zone Management Program

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CHESAPEAKE BAY SHORELINE STUDY - INTERIM REPORT

Executive Summary
December 1989

The Chesapeake Bay Shoreline Study is a cooperative project of the Commonwealth of Virginia and the Norfolk District of the U.S. Army Corps of Engineers. The project consists of three modeling and five monitoring sites located on the tributary estuarine shores of the Virginia portion of Chesapeake Bay. The purpose of the study is to examine more closely gapped offshore breakwaters and the headland concept for the abatement of estuarine shoreline erosion. These structures may represent a lower cost approach to shoreline erosion control as well as provide an "environmental edge" between what we perceive as land and marine resources.

The world-wide use of segmented or gapped breakwaters, both attached and detached, has spurred interest in the Commonwealth of Virginia. The applications of both concepts may represent an effective low cost approach to the abatement of shore erosion along hundreds of miles of estuarine shoreline. Long stretches of agricultural, wooded and unmanaged shorelines are appropriate areas for such applications.

Eight sites were selected for analysis in this study. Three sites involved the construction of offshore breakwaters which are designated modeling sites. Five sites were selected for monitoring, two of which have previously installed breakwater systems and two which exhibited crenulate-bay morphology. These sites are representative of 215 miles of estuarine shoreline in Virginia.

Analysis of the sites involved quarterly shore profiles and low level aerial photography as well as selected sediment sampling and analysis. Computer wave-modeling was performed for one site to show the effect of breakwaters on incident waves.

The definitive protective beach/breakwater system must be designed to withstand given storm conditions and the consequent surge. The main factors appear to be long, high breakwaters far enough offshore to allow for sufficient input of fill material to provide a stable protective beach and backshore.

The installation of widely spaced breakwaters to create a headland/bay situation must be done with a proper site analysis along appropriate reaches. The geomorphic expression of a shoreline, especially the fastland configuration, shows the long term response to the impinging seasonal wave climate. Evaluating the forcing by waves onto the various "natural" and anthropogenic shorelines will be the focus for further study.

CHESAPEAKE BAY SHORELINE STUDY - INTERIM REPORT

Introduction

The Chesapeake Bay Shoreline Study is a cooperative project of the Commonwealth of Virginia and the Norfolk District of the U.S. Army Corps of Engineers. The project consists of three modeling and five monitoring sites located on the tributary estuarine shores of the Virginia portion of Chesapeake Bay. The purpose of the study is to examine more closely gapped-offshore-breakwaters and the headland concept for the abatement of estuarine shoreline erosion. These structures may represent a lower cost approach to control shoreline erosion as well as provide an "environmental edge" between what we perceive as land and marine resources.

The world-wide use of segmented or gapped breakwaters, both attached and detached, has spurred interest in the Commonwealth of Virginia. The applications of this concept may represent an effective low cost approach to the abatement of shore erosion along hundreds of miles of estuarine shoreline. Long stretches of agricultural, wooded, and unmanaged shorelines may be appropriate areas for such applications.

Previous research on detached and headland (attached) breakwaters and their effects on shore morphology has been extensive (Lesnik, 1979). Much of this research has been conducted on ocean shores. The headland breakwater models were developed from physical scale models and observations of naturally occurring headlands with their adjacent crenulate, log-spiral or hook-shaped bay beaches as reported by Yasso (1965), Silvester (1970, 1974, 1976,), Silvester and Ho (1972), LaBlond (1972), Rea and Komar (1975), Finkelstein (1982), the U.S. Army Corps of Engineers (1984), Perez and Fernandez (1988), Quevauviller (1988) and Hsu et al. (1989a, 1989b). The generally stable planform, geometry or morphology of the log-spiral bay-beaches is a function of the prevailing direction of wave incidence combined with refraction and diffraction. Everts (1983) emphasized that for an equilibrium bay to form, there must be a fixed downdrift boundary.

Detached breakwaters have been examined by Toyoshima (1974), Shinohara and Tsubaki (1966), Perlin (1979) and the U.S. Army Corps of Engineers (1984). According to the Corps of Engineers, the formation of tombolos usually can be prevented if the structure length is less than the distance offshore. (A tombolo is a sandbar or spit that connects or ties a breakwater or island to the mainland or another breakwater or island.) If a detached breakwater system becomes fully attached by a consequent tombolo, the breakwater units should function more as headland breakwaters because longshore drift is essentially stopped. Unattached tombolos (cuspate spits) allow for more continuous longshore transport with less deleterious downdrift effects (U.S. Army Corps of Engineers, 1984).

In 1987, seven sites that represent different fetch exposures and shore orientations were selected for analysis. In 1988, the second year of the study, another site adjacent to an existing monitoring site was added.

Analyses of these sites involves quarterly shore profiles and low level aerial photography as well as sediment sampling and grain-size analysis. Procedures developed by Sverdrup, Munk, and Bretscheider were performed to estimate the wave climate at each site. The purpose of this report is to provide an update on the field monitoring and laboratory analysis of the Chesapeake Bay Shoreline Study from July 1987 to June 1989.

Previous Research

Shinohara and Tsubaki (1966) performed physical model tests propogating shore normal waves onto single breakwaters. They concluded that the main cause of shore change and sand movement on a beach is the diffraction of the incoming wave around the breakwater. The diffraction in turn depends on the ratio of offshore position of the breakwater to its length. The amount of sand deposition per unit area in the sheltered region behind the breakwater rapidly decreases with the increase of distance offshore.

In 1969 Toyoshima did a statistical study on 217 breakwaters in 86 locations worldwide (Toyoshima, 1974). These included single and multiple breakwaters. He stated that for gapped breakwaters, no clear factor for sand deposition and tombolo formation could be found. It was unknown which installations he studied involved beach fill. Sites with an identical ratio of breakwater length to distance offshore sometimes exhibited tombolos and sometimes not.

Perlin (1979) used a numerical model after the physical model of Shinohara and Tsubaki (1966, Figure 1). Qualitatively, the two models agree. For his model, Perlin normalized distances using linear deepwater wave length $L_o = \frac{gT^2}{2\pi}$, where g is acceleration due to gravity and T is wave period. According to Perlin, the following list of variables completely describe the problem of shore response to a single breakwater:

1) relative breakwater length,
2) relative distance of breakwater from shore,
3) relative depth of profile closure,
4) wave steepness,
5) wave angle,

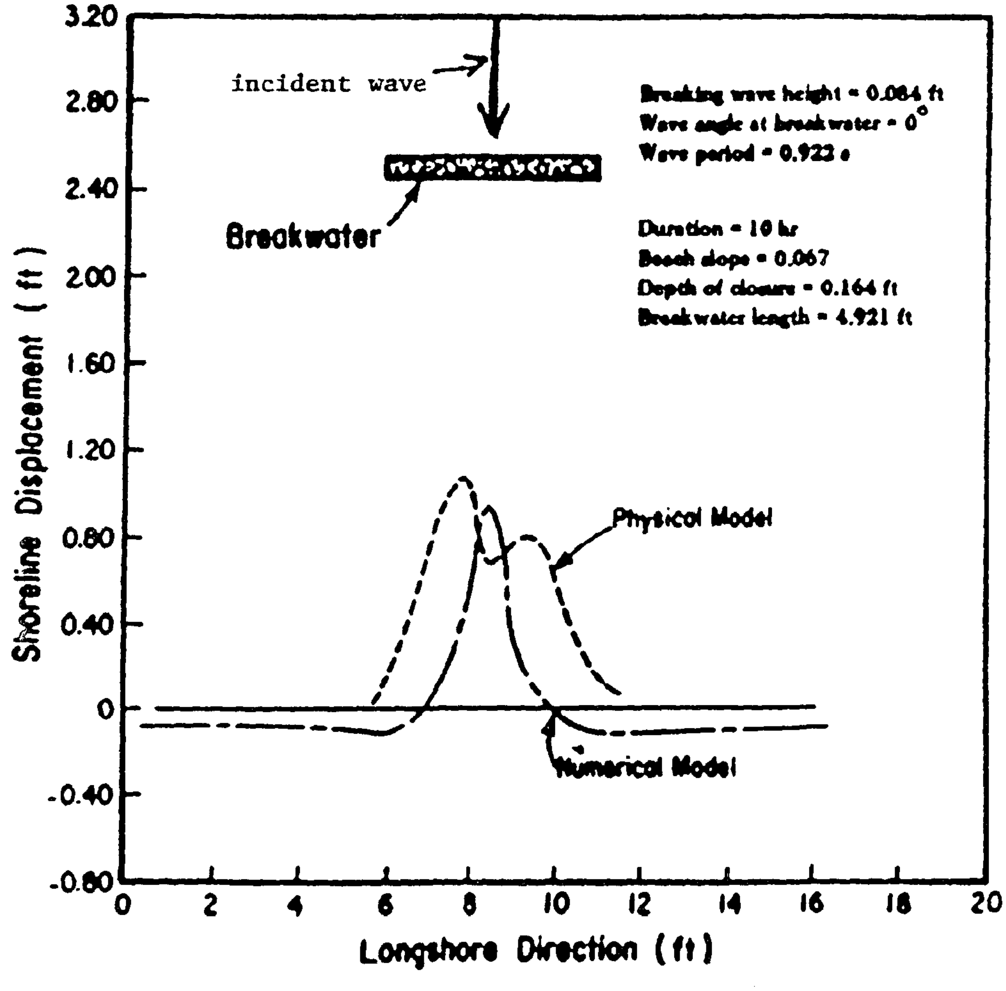


Figure 1. Simulation of the physical breakwater model by Shinohara and Tsubaki (after Perlin, 1979).

6) beach slope, and

7) number of waves through time.

In all the analyses, as the tombolo forms, adjacent shorelines erode to provide the sediment. However, as the tombolo approaches an equilibrium planform, the amount of sand it requires is reduced, and the adjacent shorelines begin to fill because the shoreline is not aligned with the wave angle (Perlin, 1979).

Perlin's analysis demonstrated some intuitively obvious ideas. As the structure is moved further offshore, it has less of an effect on the shoreline. Also, as wave steepness increases, the shoreline responds more quickly. It was also shown by this numerical model that the initial double tombolo can be a permanent feature or it can evolve into a single tombolo.

From field and model observations, Gourlay (1974) recognized the existence of wave generated currents in the lee of breakwaters and headlands. The basic mechanism producing the current was shown to be an alongshore gradient of wave set-up within the surf zone. For a given shore geometry, the alongshore current velocity primarily is determined by the deepwater wave height (Gourlay, 1976).

Rosen and Vajda (1982) concluded that a morphological and sedimentologic equilibrium is reached when the shape of the nearshore bottom and beach contour lines is such that along the sheltered beach the diffracted waves have a component of momentum flux opposed to the gradient of the mean sea level induced by radiation stress due to non-uniform wave heights along the wave fronts. This varies from what Silvester (1974) explained. According to Silvester, a state of morphologic shore equilibrium is reached when the bottom contour lines become parallel to the diffracted wave fronts. Silvester's model thus ignores the change in breaker height alongshore and only considers the Sxy component of radiation stress (which vanishes when wave crests parallel bottom contours).

Dally and Pope (1986) recognized the natural parameters most important to the design of a detached breakwater system to be those that affect wave diffraction (wave length, height, direction, and the gap width-to-wave length ratio for segmented breakwaters), natural beach slope, water-level range, native sediment-size, and available supply of sediment. They analyzed numerous in-place breakwater systems and physical model tests and found tombolo formation for single and segmented detached breakwaters generally is assured when the ratio of breakwater length (ℓ) to distance offshore (x) approaches 1.0. Conversely, to prevent tombolo formation (i.e. only spit or salient formation), breakwater length should be equal to or less than one-half the distance offshore, $\ell \leq \frac{1}{2}$ x. Tombolo formation may also be reduced by allowing waves to overtop the breakwater(s) and/or increasing breakwater permeability (Dally and Pope, 1986).

Suh and Dalrymple (1987) performed small scale model tests in a spiral wave basin for single and multiple offshore breakwaters to examine the effects of geometric parameters on the morphological change in the shore. They compared the model tests with studies reported by others and with offshore breakwaters in the field. All horizontal lengths were non-dimensionalized with respect to the offshore distance of the breakwaters from the original shoreline, X_B. Three dimensionless variables (denoted *), x_b^* (= x_b/X_B), L_B^* (= L_B/X_B), and G_B^* = (G_B/X_B), were found to be important to shore morphology, in which x_b, L_B, X_B, and G_B are the surf zone width, the breakwater length, distance offshore and the gap spacing between adjacent breakwaters, respectively. They concluded that for multiple offshore breakwaters, tombolos form when G_B^*/L_B^{*2} is about 0.5.

Silvester (1974) considered at least two fixed breakwaters or headlands in his definition of equilibrium shore. From numerous investigations of natural crenulate or log-spiral bays and physical scale models, Silvester (1974) developed a model to determine maximum bay indentation while knowing the incident wave angle, to the center line of two headland breakwaters. Suh and Dalrymple (1987) demonstrated that when the gap between two diffraction points (i.e. the ends of adjacent breakwaters) becomes approximately twice the incident wave length, the shoreline behind each breakwater responds independently. According to the U.S. Army Corps of Engineers (1984), for normal wave incidence, the diffraction effects of gapped breakwater-ends act independently when the breakwater gap is greater than five wavelengths.

Oblique incident waves approaching widely spaced breakwaters may cause an effect in the adjacent embayment. Natural headlands and their embayments have been studied by Yasso (1965), Silvester (1974), and others. The planform of the headland-bay beaches is dependent on the predominant direction of wave attack (Yasso, 1965; Silvester, 1974) (Figure 2). Headland-bay beaches often are referred to as the aforementioned crenulate or log-spiral bay beaches.

Because of the decreasing radius of plan curvature that characteristically occurs toward the headland and because the rate of decrease in radius curvature appears to be non-linear, Yasso (1965) tested the equiangular (logarithmic) spiral,

$$\frac{R_2}{R_1} = e^{\theta \cot \alpha}$$

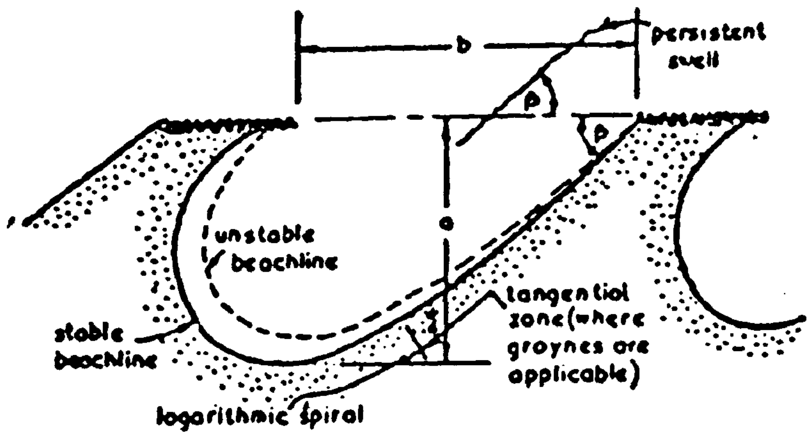


Figure 2. Crenulate shaped bay in stable and unstable conditions (after Silvester, 1976).

for goodness of fit to the plan shape of headland-bay beaches. In the equation above, $\frac{R_2}{R_1}$ is the ratio of 2 radius vectors from a log-spiral center; α is the angle between a radius vector and tangent to the wave at that point and is a constant for a given log-spiral; θ = the angle between radius vectors; and the constant e is the base of Naperian logarithms. A diagram of log-spiral nomenclature is shown in Figure 3.

Silvester (1976) recognized the difficulty in defining the equilibrium beach to the log-spiral formula. Extensive research on crenulate bays resulted in relating the equilibrium beach planform to maximum bay indentation and incident wave angle (Figure 2). Silvester divided the bay into the updrift shadow reach or logarithmic spiral and the tangential reach. The logarithmic spiral reach is affected most by wave diffraction. The tangential reach, which is slightly convex seaward or straight, is affected mostly by wave refraction.

Rea and Komar (1975), in studying log-spiral bays through numerical modeling, indicated that the shoreline will always attempt to achieve an equilibrium configuration which is governed by the patterns of offshore wave refraction and diffraction and by the distribution of wave energy flux. If the system is closed, then a true equilibrium is achieved wherein the shoreline everywhere takes on the shape of the wave crests (i.e. breaker angles are everywhere zero). If the system is not closed and sediment continues to be transported to the downdrift end of the model and further, then equilibrium occurs where the breaker angles are precisely those required to transport the sediment eroded from the updrift section of beach. Under this definition of equilibrium the shoreline continues to erode but retains its overall shape (Rea and Komar, 1975).

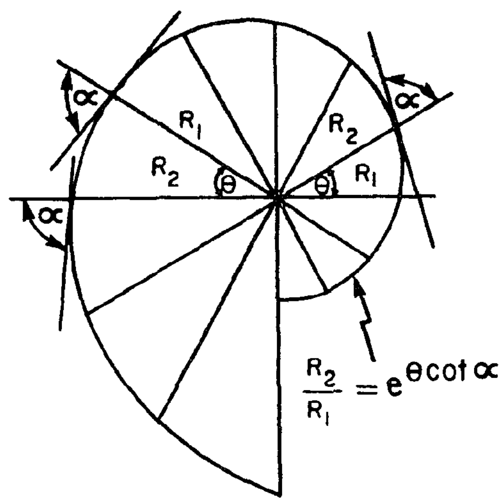


Figure 3. Definition sketch of logarithmic spiral (after Yasso, 1965).

Everts (1983) recognized the difficulty in using a logarithmic spiral shape in that establishing the center location of the spiral must be done by trial and error. He noted for an equilibrium, crenulate-shaped bay to form, there must be a fixed downdrift boundary. Without one, the rate of sediment loss will not decrease progressively with time after headland or breakwater construction. Only with a fixed boundary will the alongshore length of the bay be controlled and the total volume loss be fixed. However, the downdrift boundary does not have to be a littoral barrier. It must, though, provide a fixed limit for the bay such that the angle between the equilibrium-tangent-sector-alignment and the pre-construction shoreline becomes constant at the downdrift boundary until equilibrium conditions are reached (Everts, 1983).

Perez and Fernandez (1988) found that the log-spiral equilibrium formula is applicable to over 30 pocket beach locations along the Mediterranean coast of Spain. Spanish pocket beaches closely fit the following equation:

$$S = 25 + 0.85\ A$$

where S is the gap between headland breakwaters and A is the depth of the pocket beach.

Hsu et al. (1989) determined that defining bay curvature through the log-spiral method was not precise and should be replaced by some new relationships. These new relationships revolve around what Silvester calls a static equilibrium bay (Figure 4). The line joining the point of diffraction to the downcoast limit of the bay (R_o) is termed the "control line" and its angle to the incident wave crests is the obliquity of the waves (β), which is the only input variable that determines the bay shape. This angle is the same as that between R_o and the downcoast tangent to the

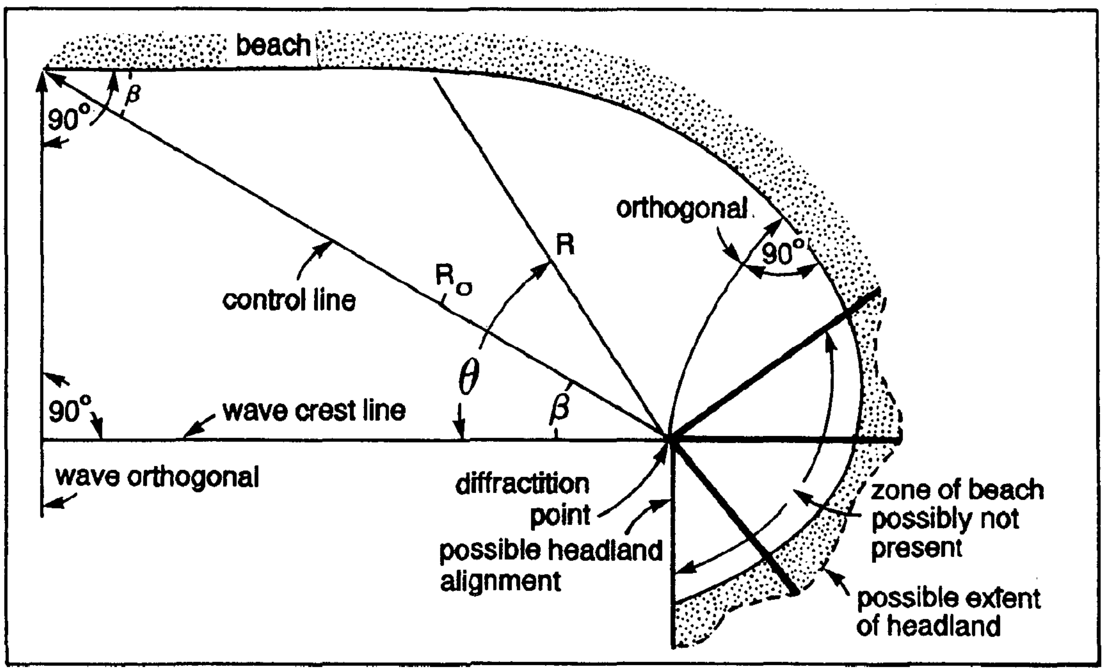


Figure 4. Parameters of the static equilibrium bay (after Hsu et al., 1989).

beach when the bay is in static equilibrium. From the definition sketch in Figure 4, it is seen that the variables involved in this new presentation are an arc of length R angled θ to the wave crest line, which is assumed parallel to the tangent at the downcoast limit of the beach (Hsu et al., 1989). Hsu et al. (1989) admitted that the log-spiral is still useful as a secondary check but his recent reassesement should be carefully studied.

Data Collection

Field Methods

The eight project sites for the Chesapeake Bay Shoreline Study are:

Modeling Sites

Breakwaters

1. Chippokes State Park, James River, Surry County (CHP)
2. Hog Island Breakwaters, James River, Surry County (HI2)

Headland

1. Hog Island Headlands, James River, Surry County (HIH)

Monitoring Sites

Breakwaters

1. Drummonds Field, James River, James City County (DMF)
2. Parkway Breakwaters, York River, York County (NPS)
3. Waltrip, James River, James City County (WAL)

Headlands

1. Summerille, Potomac River, Northumberland County (SUM)
2. Yorktown Bays, York River, York County (YB)

Figure 5 shows the locations of the sites.

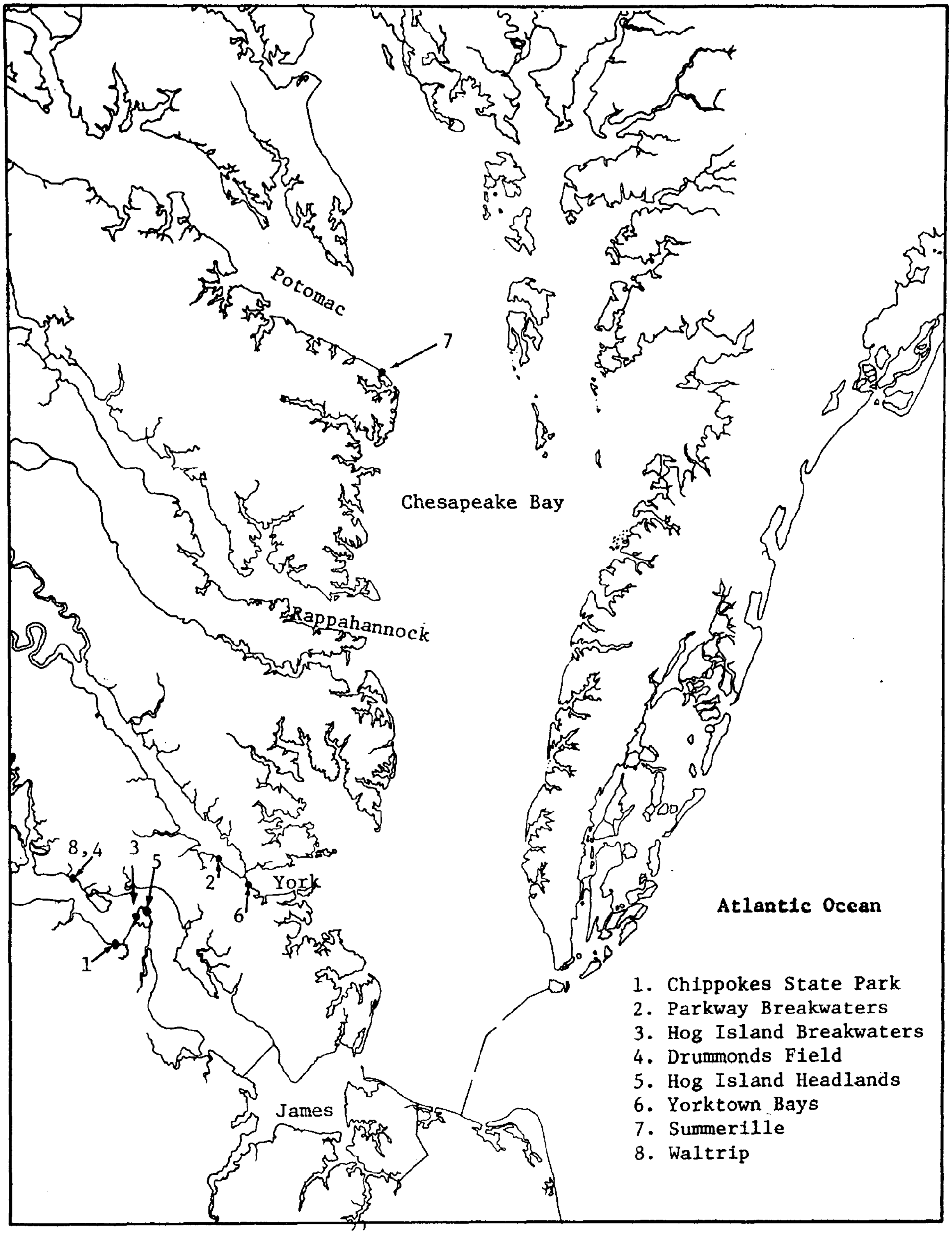


Figure 5. Virginia Chesapeake Bay and its tributaries with project site locations.

Shore-parallel baselines were established for each site with profile distances and elevations determined using stadia and level. The position and spacing of the profiles were site specific. Additional profiles were established at the breakwater sites in order to measure more accurately the changes in shore orientation. The long curvilinear shores at two sites, Summerille and the Yorktown Bays, had less closely spaced profiles. Table 1 is the schedule of profiling and aerial photography.

Initially, aerial photography was done during each phase of the project at 500, 1,000 and 2,000 feet. Later, in March 1989, this was changed to 750 and 1,500 feet so that an even scale of 1 inch 100 feet and 1 inch = 200 feet respectively could be used directly. The photographs were used along with the profile data to create a base map for each site upon which the baseline, profile locations, the shoreline and banks, the breakwaters and/or headlands could be drawn to scale. Some profile data and aerial photography that had been acquired before the Chesapeake Bay Shoreline Study were incorporated in the analysis.

Surface sediment samples were collected from the beach and nearshore areas along selected profiles at each site. Selected surface samples were analyzed for percent of gravel, sand, and mud utilizing the Rapid Sand Analyzer (RSA) at VIMS. The graphic mean, median and standard deviation (Sd) were the statistical parameters used to evaluate the sediment samples. The standard deviation is used to determine the sorting of each sample.

Of the five breakwater sites, three, Chippokes, Hog Island Breakwaters and Parkway Breakwaters, are similar in that the breakwaters were initially located at or near mean low water (MLW). All of the breakwater sites involved structures with some degree of tombolo

Table 1. Profiling and Aerial Photography Schedule by Month and Day

	1988						1989					
	Phase I			Phase II			Phase III			Phase IV		
Site	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun
<u>CHP</u>												
33 profiles												
baseline = 839 ft												
profiles			21			14			28			8
aerial photos			15			19			29		25	
<u>NPS</u>												
27 profiles												
baseline = 748 ft												
profiles			14			6			17			2
aerial photos			15			19			29		25	
<u>HI2</u>												
61 profiles												
baseline = 1275 ft												
profiles			7		22				16			6
aerial photos			15			19			29		25	
<u>DMF</u>												
35 profiles												
baseline = 1674 ft												
profiles			13			8			30			15
aerial photos			15			19			29		25	
<u>WAL</u>												
17 profiles												
baseline = 447 ft												
profiles			27			20			20		26	
aerial photos			15			19			29		25	
<u>HIH</u>												
30 profiles												
baseline = 2400 ft												
profiles			6	20		6			17			8
aerial photos			15			19			29		25	
<u>YB</u>												
21 profiles												
baseline = 357 ft												
profiles			28		17	19			15		23	
aerial photos			15			19			29		25	
<u>SUM</u>												
12 profiles												
baseline = 863 ft												
profiles			2			5			22			5
aerial photos						19			29		25	

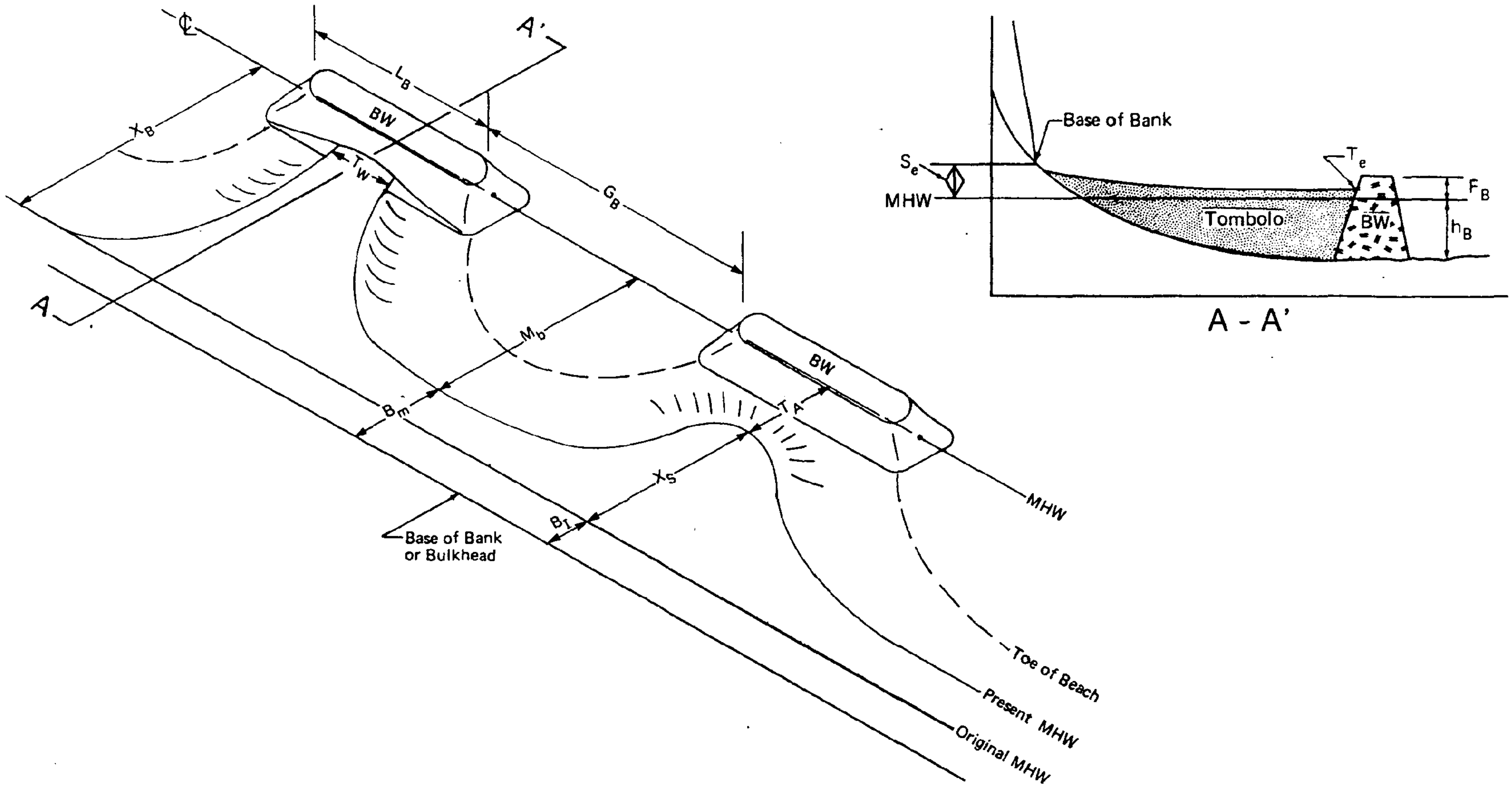
formation. Chippokes and Parkway Breakwaters had no beach fill added, whereas Hog Island Breakwaters and Drummonds Field did. Waltrip was constructed in 1988 with beach fill added and is located adjacent to Drummonds Field.

Of the three headland sites, only Hog Island Headlands was constructed for this project. Both Summerille and Yorktown Bays have existed as a log-spiral bay and pocket beach for over 10 years. The designation of breakwater and headland is somewhat arbitrary since all the study sites function to a degree as headland breakwaters. The main difference is that the designated headland sites have much wider gaps or bays relative to breakwater (headland) length.

The parameters that will be discussed are depicted in Figure 6. Minimum beach width, B_m, is considered to be the parameter around which a protective breakwater system should be designed. This includes the point at the maximum indentation (M_b) between structures which is the most vulnerable area to wave attack under storm conditions. The backshore beach elevation (Se) is also an important parameter. High, wide beaches offer greater bank protection during storm events. Therefore, the level of protection that the embayed beaches provide for a given storm condition must be defined. Beach surveys immediately after a moderate northeaster in April 1988 provided some insight into how several bay beaches responded to storm conditions.

Wave Climate

The wave climate along the tributary estuaries of the Virginia Chesapeake Bay Estuarine System is fetch limited. Six of the eight sites in this study have average fetches of 2 to 3.5 nautical miles. Two sites,



L_B - Breakwater crest length

G_B - Breakwater gap

X_B - Distance offshore CL breakwater to original MHW

h_B - Height of breakwater from bottom at CL to MHW

F_B - Breakwater freeboard, MHW to crest

M_b - Maximum bay indentation, CL breakwater to MHW

T_e - Tombolo elevation in lee of breakwater $\pm$ MHW

S_e - Backshore elevation at base of bank

B_I - Initial beach width, base of bank to MHW

B_m - Present beach width, base of bank to MHW

T_A - For unattached tombolo, MHW to CL of breakwater

T_W - For attached tombolo, tombolo width at MHW

X_S - Salient length

Figure 6. Perspective sketch of breakwater parameters.

the Yorktown Bays and Summerille, have fetches of approximately 10 nautical miles. The seasonal wave climate favors northerly winds in the winter and southwesterly winds in the summer (Figure 7). Mean seasonal winds generate limited waves across the rivers.

The wave climate at each site was estimated using procedures outlined by the Sverdrup-Munk-Bretscheider (SMB) method as modified by Camfield (1977). An average fetch analysis of wind conditions mostly likley to affect each site was done using methods outlined in the U.S. Army Corps of Engineers Shore Protection Manual (SPM) (1984). For the SMB analyses, bathymetric transects for each wave direction were created by segmenting the bottom contours into average depths. Nearshore depths were segmented more closely to better approximate the actual bottom slope.

The SMB method was performed for several conditions of storm surge and wind speed. The resulting wave parameters include wave height, period and length. These parameters may then be used as incident wave conditions for the VIMS modified RCPWAVE model developed by the U.S. Army Corps of Engineers (Ebersole et al., 1986) or manual wave refraction analysis across a nearfield (nearshore) bathymetric grid. This in turn will provide the onshore angle of wave approach for various wind directions, wind speeds and storm surges.

For comparative purposes, only the results of the SMB analyses are depicted in this interim report. Also, the effects of tidal currents are excluded. Tidal currents will play a critical role in the wave refraction process, especially under storm conditions.

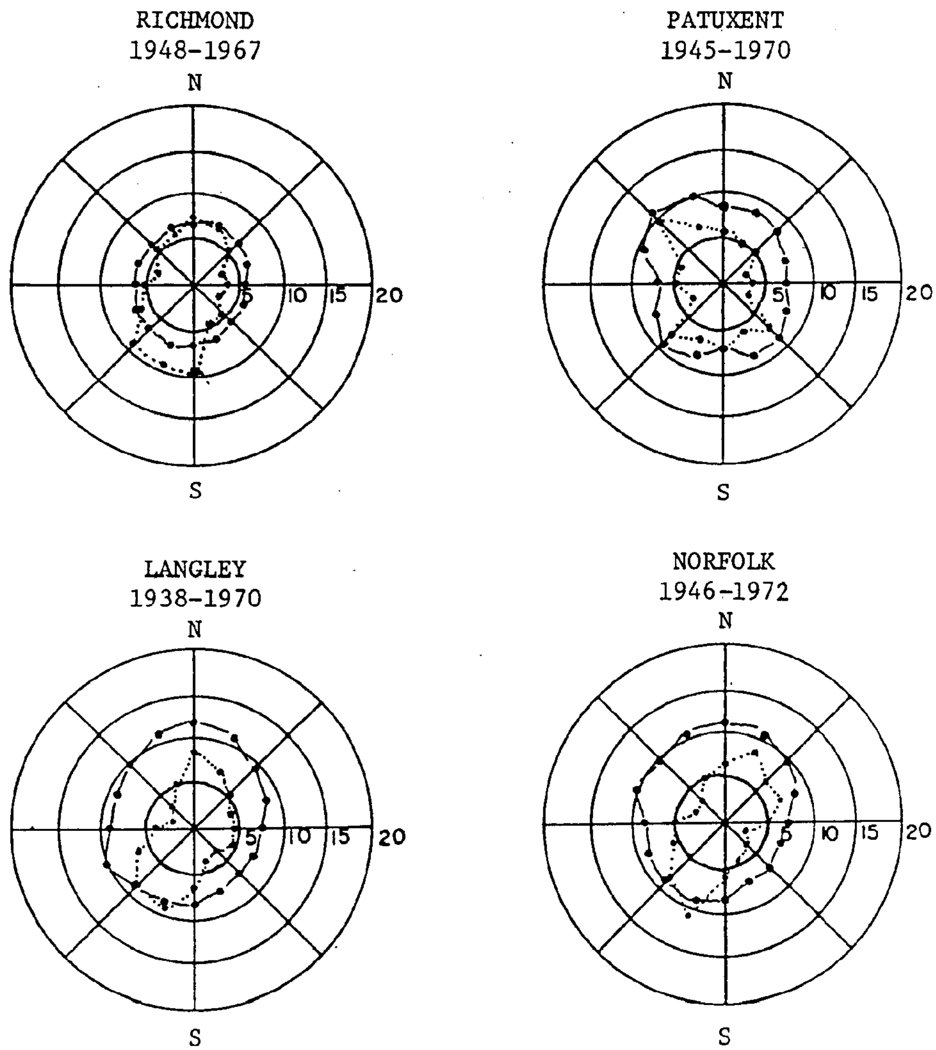


Figure 7. Long term wind roses for Richmond, Patuxent, Langley, and Norfolk.
•········• Frequency of occurrence(%)
•——• Average velocity(kts)
(Hardaway et al., 1984)

Shore Morphology

Modeling the effects of wave/current interaction is a complex procedure. At this point our best estimate of long term effects of wave climate (especially angle of wave approach) on a given site can be determined from an evaluation of a shoreline's evolution to its present state. Shorelines in the Chesapeake Bay Estuarine System, being composed of varying lithologies, erode at different rates. Headland-bay situations often evolve when one section of shore is artifically stabilized and the adjacent shoreline is not. The long-term direction of wave aproach is reflected in the orientation of the tangential section of the the eroded embayment (Figure 2). Headland-bay shorelines have evolved over the past 15 to 50 years at the Yorktown Bays, Summerille, Drummonds Field and at the Hog Island Headlands. Aerial photography was used to determine the evolution of these sites and thus net direction of wave approach.

Storm Events

The frequency of storm surges in the Chesapeake Bay was reported by Boon et al. (1978). At Hampton Roads, the storm surge for extratropical (i.e. northeaster) storms for a 10-year and a 50-year storm are +3.2 feet and +3.8 feet above MHW respectively. Extratropical and tropical storms with the associated storm surges and increased wave energy are the main forces causing movement of beach sand and shoreline erosion. Table 2 lists the major storm events experienced in southeastern Virginia between 1956 and 1978.

A period of sustained northeasterly winds was experienced in southeastern Virginia between April 11 and April 13, 1988. According to the National Oceanic and Atmospheric Administration (NOAA), the average

Table 2. Occurrence of Major Storms in Southeastern Virginia From 1956-1978

Storm	Date	Storm Surge (ft)	Wind Speed (kn)	Direction
	11 Jan 1956	3.4	33	NE
	11 Apr 1956	4.3	62	N
	03 Nov 1956	2.0	29	NE
	28 Feb 1957	2.4	33	NE
	08 Mar 1957	2.2	27	NE
	01 Nov 1957	2.7	28	NE
	25 Jan 1958	2.3	44	E
	01 Feb 1958	2.2	30	W
	19 Mar 1958	2.2	21	NE
	27 Mar 1958	2.6	20	N
	11 Dec 1958	2.1	27	NE
	29 Dec 1958	2.3	38	E
	12 Apr 1959	2.5	45	NE
	19 Dec 1959	2.1	29	N
	31 Jan 1960	3.0	42	NE
	13 Feb 1960	2.3	49	NE
	03 Mar 1960	2.4	52	E
	12 Dec 1960	2.0	40	W
	16 Jan 1961	2.0	13	W
	08 Feb 1961	2.4	27	NE
	22 Mar 1961	2.2	33	E
	28 Nov 1961	2.0	23	NW
	28 Jan 1962	2.2	37	NE
Ash Wed	07 Mar 1962	5.6	41	NE
	22 Mar 1962	2.4	20	N
	03 Nov 1962	2.5	33	N
	26 Nov 1962	3.3	41	N
	08 Feb 1963	2.3	30	NE
	06 Nov 1963	2.4	38	E
	04 Jan 1964	2.0	28	W
	12 Jan 1964	2.6	42	E
	12 Feb 1964	2.0	32	E
Cleo	01 Sep 1964	1.0	42	ESE
Dora	18 Sep 1964	0.3	61	NE
Gladys	23 Sep 1964	2.3	44	N
Isabell	16 Oct 1964	2.6	50	NE

Table 2 (continued)

Storm	Date	Storm Surge (ft)	Wind Speed (kn)	Direction
	16 Jan 1965	3.9	35	NE
	22 Jan 1965	3.0	36	E
	29 Jan 1966	3.6	37	E
Alma	13 Jun 1966	1.0	40	N
	24 Dec 1966	2.3	31	NE
	07 Feb 1967	2.6	33	NE
Doria	16 Sep 1967	3.4	55	N
	12 Dec 1967	2.0	30	E
	29 Dec 1967	2.0	31	W
	14 Jan 1968	2.3	33	E
	08 Feb 1968	2.6	30	NE
Gladys	20 Oct 1968	1.3	46	NE
	10 Nov 1968	4.3	34	N
	12 Nov 1968	2.6	47	NE
	02 Mar 1969	5.9	40	N
	02 Nov 1969	2.6	36	NE
	10 Nov 1970	2.6	22	SE
	16 Dec 1970	2.0	31	E
	27 Mar 1971	2.8	45	NE
	06 Apr 1971	4.0	44	NE
	19 Oct 1972	-	34	N
	11 Feb 1973	3.5	44	N
	21 Mar 1973	3.1	28	N
	02 Mar 1975	2.2	22	SSE
	14 Oct 1977	2.6	29	NE
	30 Oct 1977	2.3	24	NE
	20 Dec 1977	-	-	-
	28 Apr 1978	4.6	39	NE

(W.S. Richardson, U.S. Weather Service, personal communication, 1979)

Revised from Senate Document No. 4, Report of the Coastal Erosion Abatement Commission, 1979.

peak wind speeds and directions at Norfolk International Airport were as follows:

Date	Average Speed (mph)	Average Direction	Peak Speed (mph)	Peak Direction
11 April	10.6	NE	23.0	E
12 April	23.4	NE	47.0	NE
13 April	28.3	NNE	51.0	NE

Storm surges measured at VIMS ranged from about +1.0 foot MHW on April 11, 1988 to about +3.0 feet MHW on April 13, 1988. Field observations were made at Yorktown Bay 1, Parkway Breakwaters, Drummonds Field, Chippokes, Hog Island Breakwaters and Hog Island Headlands on April 13, 1988, during the peak of the storm. The results of these observations are shown in Table 3. The observed wave parameters were measured just outside the line of breakwaters or just before breaking.

In 1989, two coastal storms passed through southeastern Virginia. The first storm occurred on February 24 in the form of a blizzard with wind gusts to 50 mph from the northwest to the north northeast. Winds at Norfolk International Airport averaged 26.2 mph (NOAA, 1989) and a storm surge of only 1.5 feet above MHW was observed. The second storm occurred on March 6 through 9 as a moderate northeaster with average winds of about 24 mph and gusts of 40 to 45 mph (NOAA, 1989). This storm mostly affected the ocean coast of Virginia where most of the property damage occurred. There was about a 2-foot storm surge in the Chesapeake Bay but very little wind and wave action was experienced as compared to the April 1988 storm.

The following section is a discussion of the monitoring sites and the types of data that is being collected and reduced. Examples of data trends are displayed and an attempt has been made to provide some insight into the results we see at this stage of the study. All the figures are found at the end of the discussion for each site chapter.

Table 3. Wave Observations, Northeaster of 13 April 1988

Site	Wave Angle (TN) (degrees)	Wave Height (feet)	Wave Period (seconds)
YB1	65-70	2.0-2.5	3.5-4.0
NPS	40	1.0-1.5	2.0
HI2	335	1.0-1.5	2.0-2.5
HIH	25	1.0-1.5	2.0-2.5
CHP	15	1.0-2.0	2.5-3.0

SITES

Chippokes State Park, James River, Surry County

The gapped breakwater system at Chippokes State Park is located on Cobham Bay (Figure 8). Chippokes is a recreational and historic state park as well as a "model" farm. The site lies within an estuarine reach of the James River between College Run and Lower Chippokes Creek. The reach is characterized by high (40 ft), eroding, fastland banks which give way to low fastland banks toward each bounding drainage. The high banked shore at Chippokes faces almost due north.

Cobham Bay appears to be the geomorphological remnant of the outside bank of a meander of the ancestral James River. Erosion of the bank is driven by wind and waves from the northeast and northwest. The high banks are composed of a lower unit of shelly, fossiliferous, fine to coarse sand overlain by an upper layer of slightly muddy, fine to medium sand. Net transport here is eastward but with seasonal fluctuations and onshore-offshore movement.

The preconstruction beach at Chippokes was a curvilinear strand of sand about 25 feet wide from MHW to the base of the bank. The beach itself consists of a fine to coarse, well sorted, shelly sand derived from the eroding bluff.

Wave Climate

The Chippokes breakwater system faces almost due north with an average fetch of 2.4 nautical miles. Long fetches of 5.0 and 8.0 nautical miles occur to the north northeast and northwest, respectively. Strong seasonal winds from the north and northwest tend to force beach sediments to the east (Figure 9A). During northeast storm events (i.e. April 1988), waves approach from the north to north northeast with breaking wave heights of 1.4 to 1.8 feet (Figure 9B).

Design and Construction

The goal at Chippokes was to design a system which would permit a tombolo to form utilizing the existing volume of sand on the beach, such that, with time a stable backshore would develop and protect the base of the high banks. A system of six breakwater units with a length to gap ratio of 1:1.5 was designed (Figures 10A and 10B). The crest lengths are 50 feet and gaps are 75 feet. The centerline of the breakwaters is approximately 30 feet from the initial MHW line and the crest width of each breakwater is 4 feet.

Construction of the breakwater system took place during June 1987. A road had to be cut down the bank to provide access for the equipment. Subsequent rains washed out the road several times, thus providing additional material to the beach system. Rock for construction of the rubble mound breakwaters, as depicted in the SPM (U.S. Army Corps of Engineers, 1984), was trucked in and dumped over the bank behind the site for each breakwater unit. The rock was then placed with a large, tracked backhoe.

Shore Changes

Figure 11 shows shoreline changes from June 1987 to March 1989. The position of the MHW line was used to track beach changes at each site. Sand began accumulating and migrating toward each breakwater unit as cuspate spits formed almost immediately after construction. The shore behind breakwater number 1 showed the quickest response. The characteristic double spits (Perlin, 1979, Rosen and Vajda, 1982) evolved behind each structure by September 1987. By February 1988 the double salients or saddles had coalesced into a single, attached tombolo with a swale between the saddles. Sediment for the tombolos was derived from the adjacent embayments. This is most evident in Bays A, B, and C. Tombolos eventually attached symmetrically to the lee of each breakwater.

By February 1988 all the bays showed signs of filling. Accretion of sand on the west end of the system and a marked loss of sand on the east end was apparent. One would infer a net west to east movement of sand along this portion of the reach.

In April 1988, northeast winds blew continuously across the James River and Cobham Bay for three days. Post-storm surveys showed a general decrease in the intertidal beach slope in the center of each embayment and erosion along the base of the bank. There was a corresponding increase in tombolo elevation (Te) behind each breakwater but the overall tombolo widths (Tw) were reduced. Figure (12B) shows the typical profile response to the storm in the lee of a breakwater. Material contributing to the increased tombolo elevation came partially from the eroded embayment beaches and partially from runoff down the upland banks. There did not appear to be any significant offshore movement of beach material. Mid-bay profiles indicate no beach shifts beyond the limits of the breakwaters (Figure 12A).

Figure 13 depicts how selected parameters for a typcial bay and breakwater changed through time. Changes due to the April 1988 storm were seen in backshore elevation (Se), bay beach width (Bm), tombolo elevation (Te), and tombolo width (Tw). Bay indentation (Mb) remained relatively constant after the storm and through to June 1989. This would infer that the position of MHW moved little in the embayments during the same period. Table 4 shows the status of the Chippokes breakwater system's parameters.

Sediment samples taken along profile 13 at MHW and step show a noticable response to the April 1988 storm (Figure 14). Beach sands were finer and better sorted immediately after the storm. A general change to a coarser less sorted beach material followed.

The net rate of volumetric changes in the beach between -2 feet and +2 feet is shown in Figure 15. From July 1987 to Feb 1988 high rates of erosion and accretion were noted in the embayments and behind each breakwater, respectively. This was interpreted as being the initial shore response to the breakwater installation. A slow accretionary trend followed throughout the Chippokes breakwater system except immediately updrift and downdrift where accretion and erosion, respectively, were noted. Sand began bypassing breakwater number 1 by September 1988 and infilling of bay A began. This trend has continued through March 1989.

The overall shape of each embayment has remained constant. Slight shifts are observed in beach postion in response to more oblique northwest winds but a general symmetrically curvilinear planform persists.

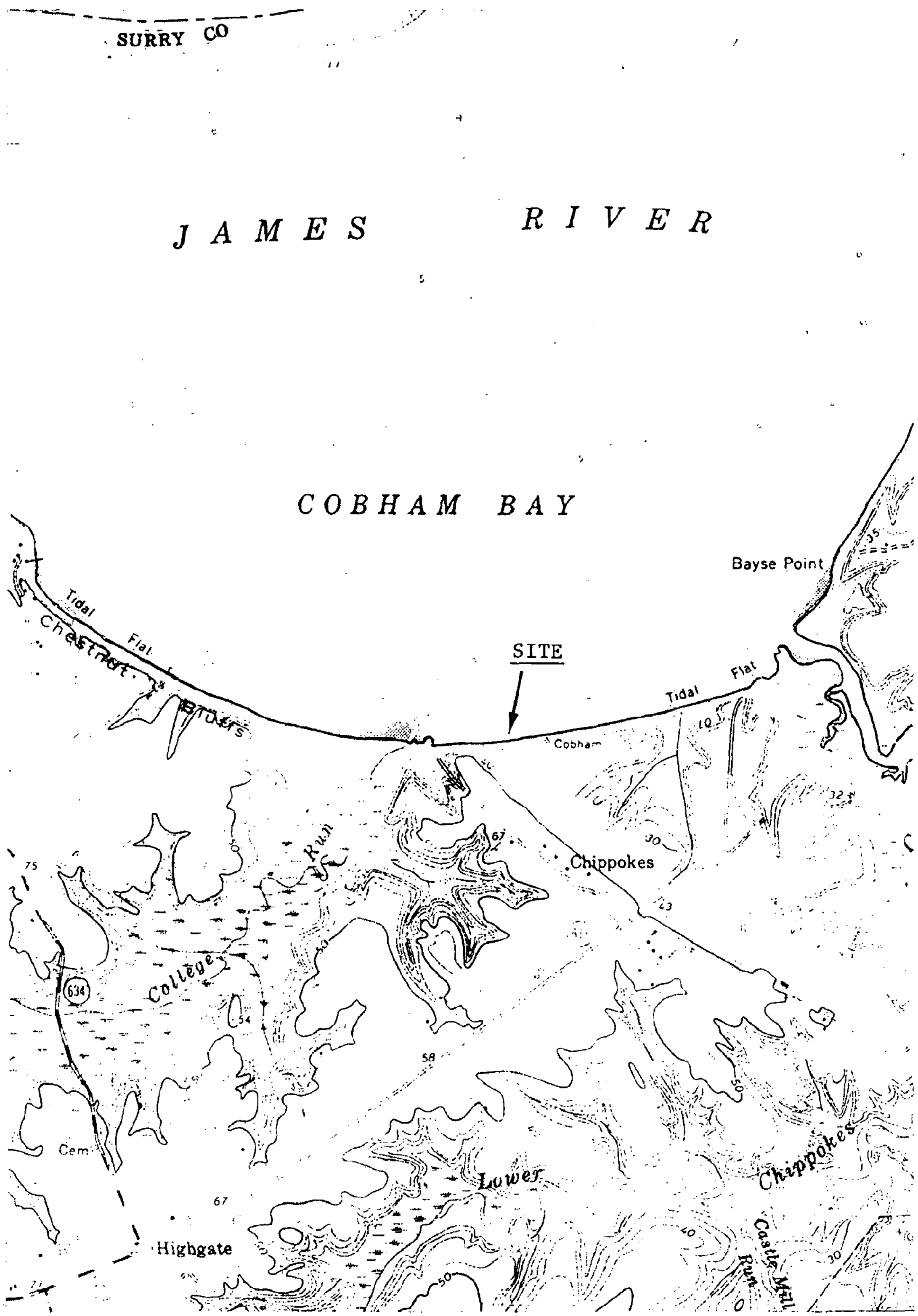
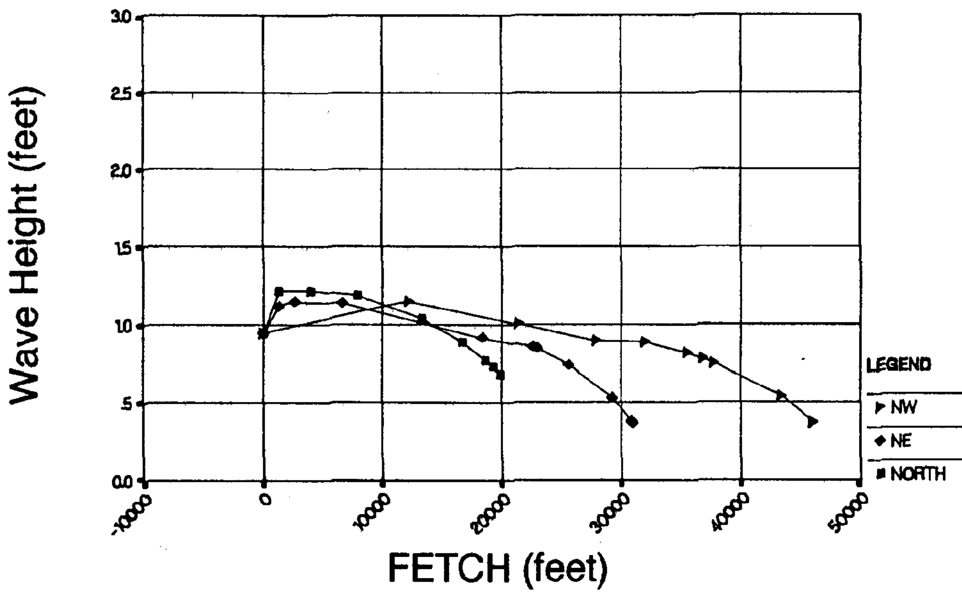


Figure 8. Chippokes State Park, James River, Surry County.
From Hog Island 7.5 minute quadrangle.
Scale: 1 inch = 2,000 feet.

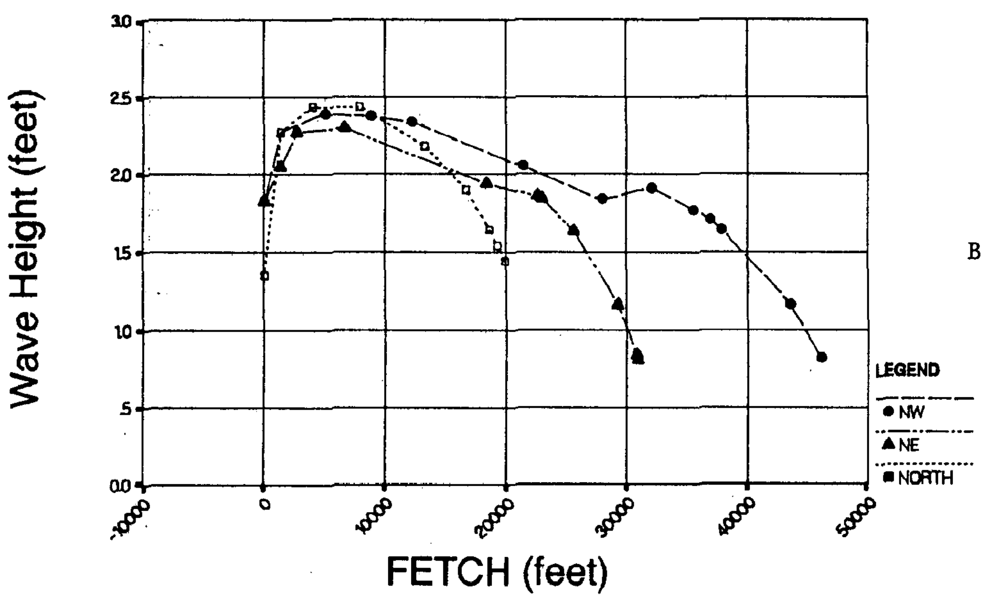
SURGE AT 2 FEET ABOVE MLW WITH WIND 20 MPH.

CHP



A

SURGE LEVEL 4 FEET ABOVE MLW WITH WIND 40 MPH

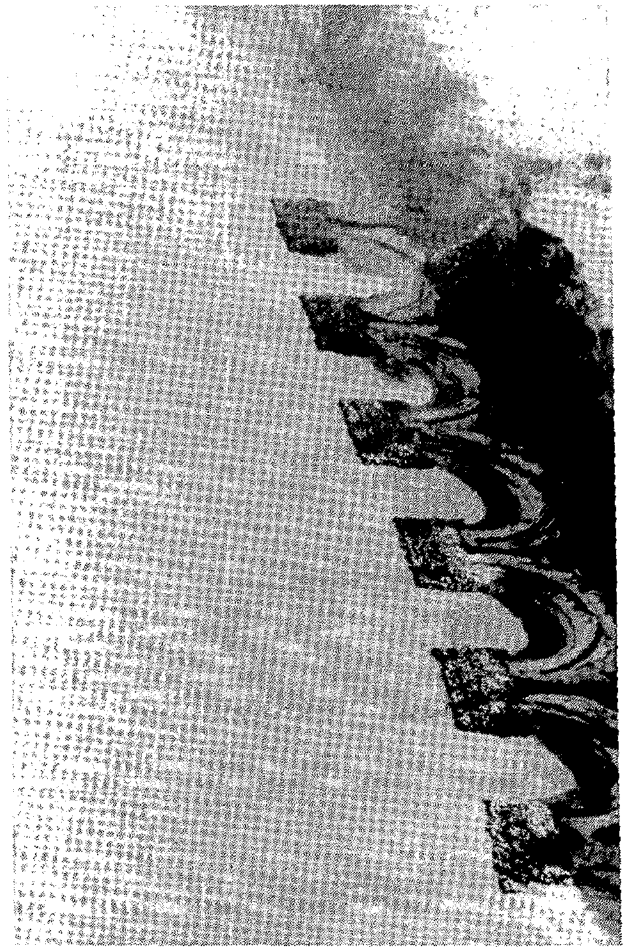


B

Figure 9. Chippokes State Park - Wave Climate.

Figure 10A. Chippokes State Park - aerial view, looking east.

Figure 10B. Chippokes State Park - ground view, looking east. Breakwater number 2 in the foreground.



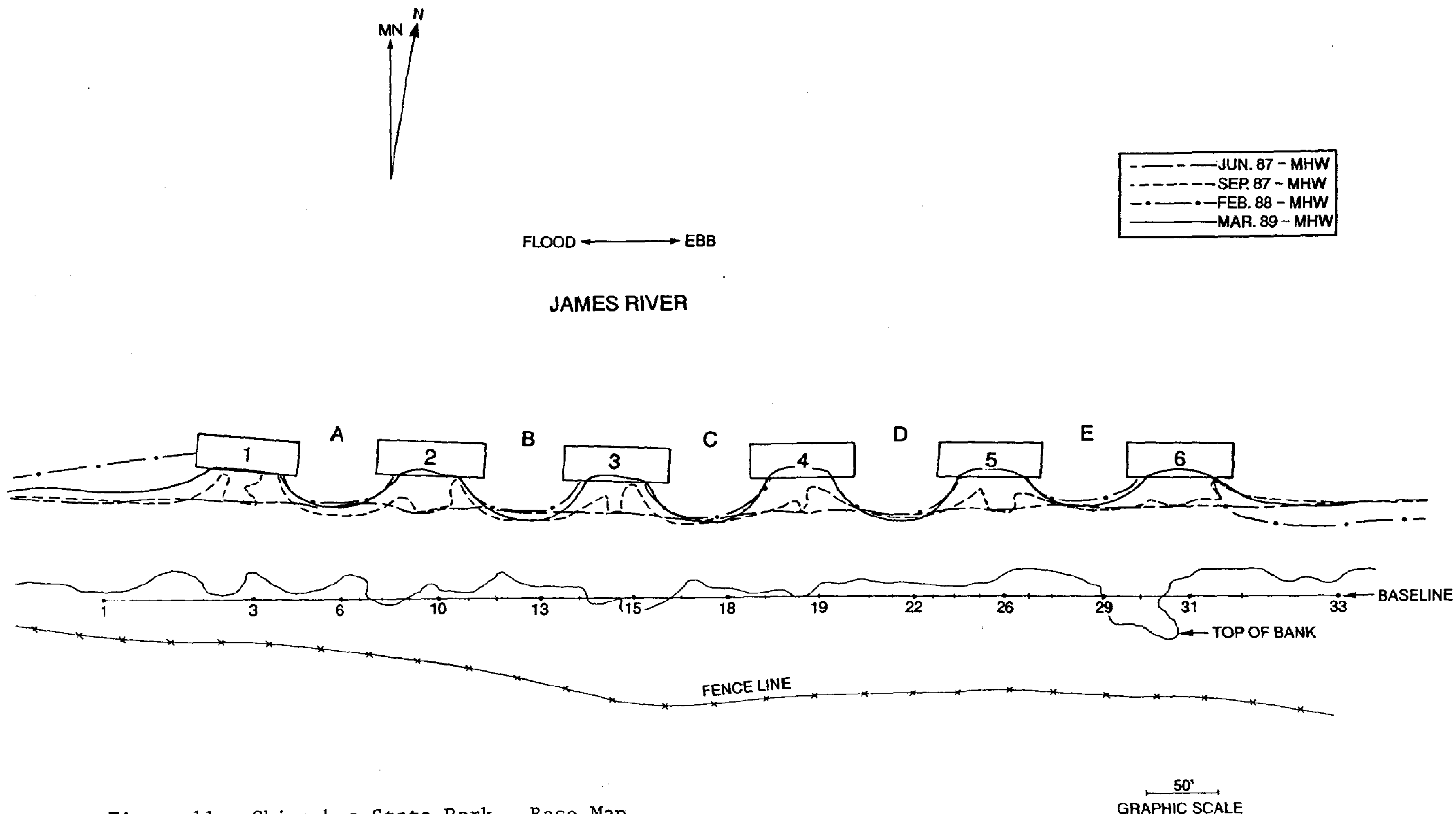


Figure 11. Chippokes State Park - Base Map.

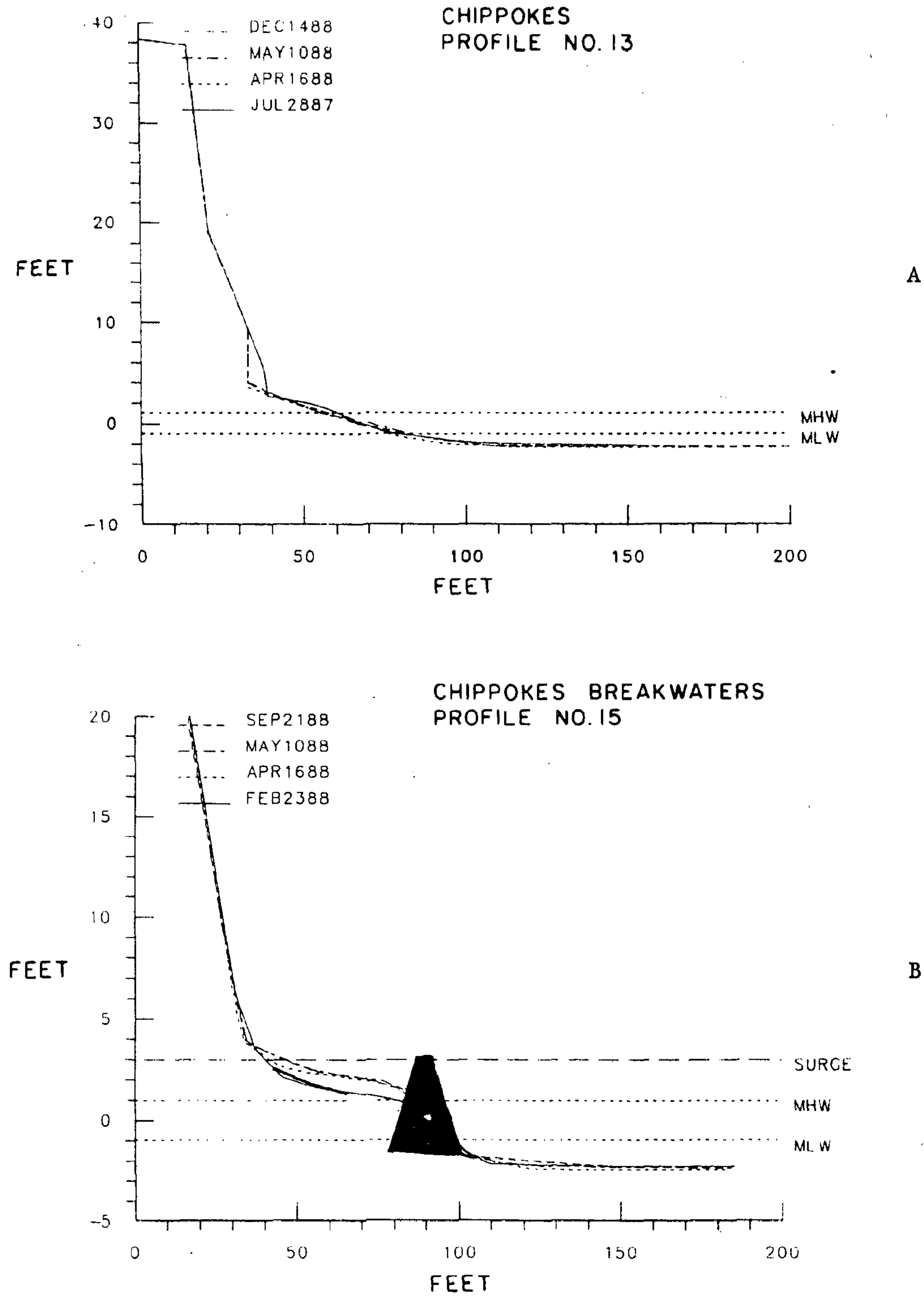
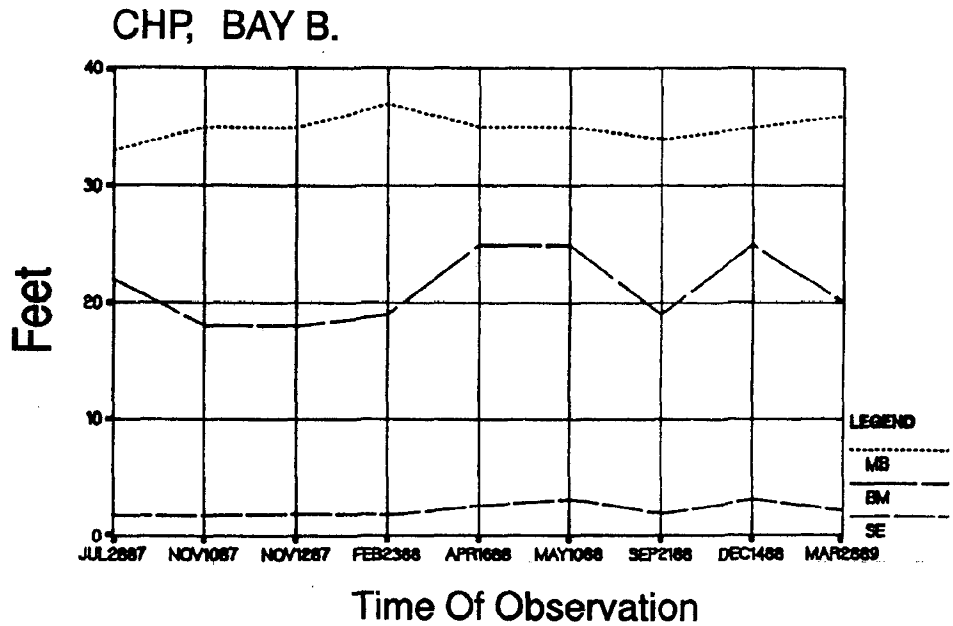


Figure 12. Chippokes State Park - Representative Profiles.



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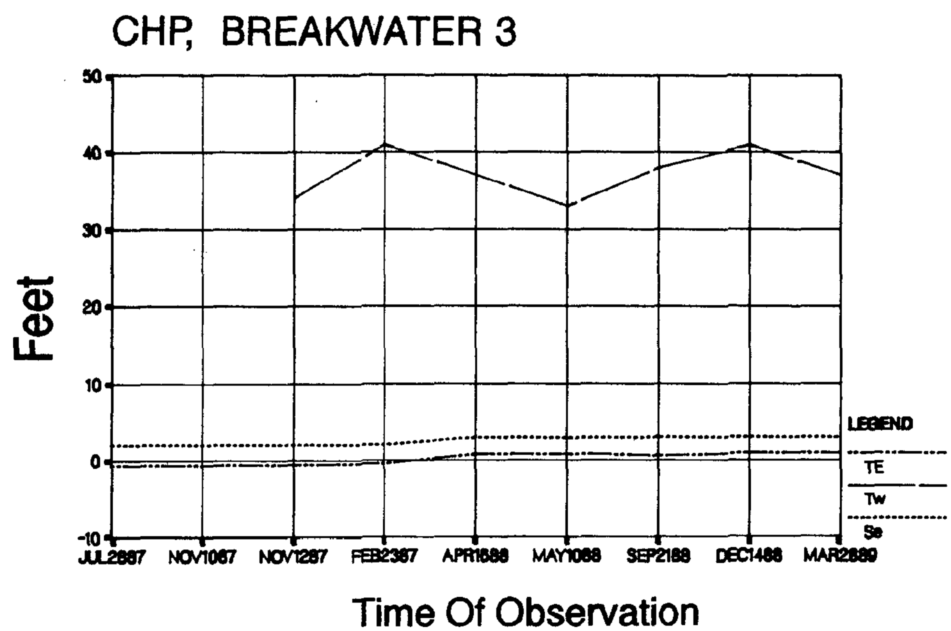


B

Figure 13. Chippokes State Park - Representative Parameters.

Table 4. Parameters for Chippokes State Park*
March 1989

Breakwater/Bay	L_B	G_B	X_B	h_B	F_B	M_b	T_e	S_e	B_I	B_M	T_A	T_W
Updrift								2.4	23	32	N/A	
Breakwater 1	50		30	2.5	2.2		0.8	3.9	20	43	N/A	54
Bay A		75				29		2.6	20	27	N/A	
Breakwater 2	50		30	2.7	2.3		1.3	2.8	20	46	N/A	41
Bay B		75				36		2.1	21	20	N/A	
Breakwater 3	50		33	2.9	2.1		1.0	3.1	16	48	N/A	37
Bay C		75				40		1.8	13	14	N/A	
Breakwater 4	50		35	2.6	2.1		0.6	2.9	15	45	N/A	40
Bay D		75				38		0.9	10	8	N/A	
Breakwater 5	50		35	2.8	2.0		0.7	3.0	12	42	N/A	36
Bay E		75				34		2.4	10	22	N/A	
Breakwater 6	50		33	2.6	2.1		1.1	4.1	10	43	N/A	60
Downdrift								4.2	14	26	N/A	

* All dimensions in feet.

L_B - Breakwater crest length

G_B - Breakwater gap

X_B - Distance offshore CL breakwater to original MHW

h_B - Height of breakwater from bottom at CL to MHW

F_B - Breakwater freeboard, MHW to crest

M_b - Maximum bay indentation, CL breakwater to MHW

T_e - Tombolo elevation in lee of breakwater + MHW

S_e - Backshore elevation at base of bank

B_I - Initial beach width, base of bank to MHW

B_m - Present beach width, base of bank to MHW

T_A - For unattached tombolo, MHW to CL of breakwater

T_W - For attached tombolo, tombolo width at MHW

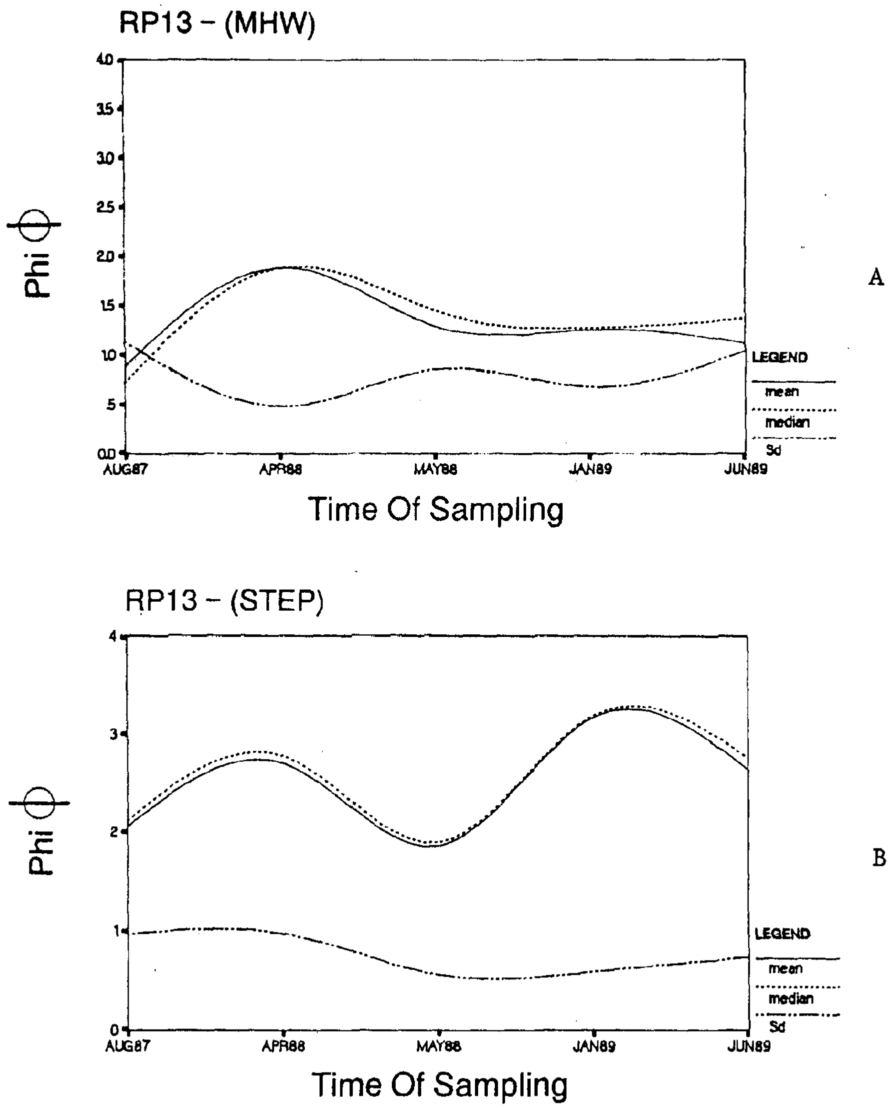


Figure 14. Chippokes State Park - Representative Beach Sediment Analysis.

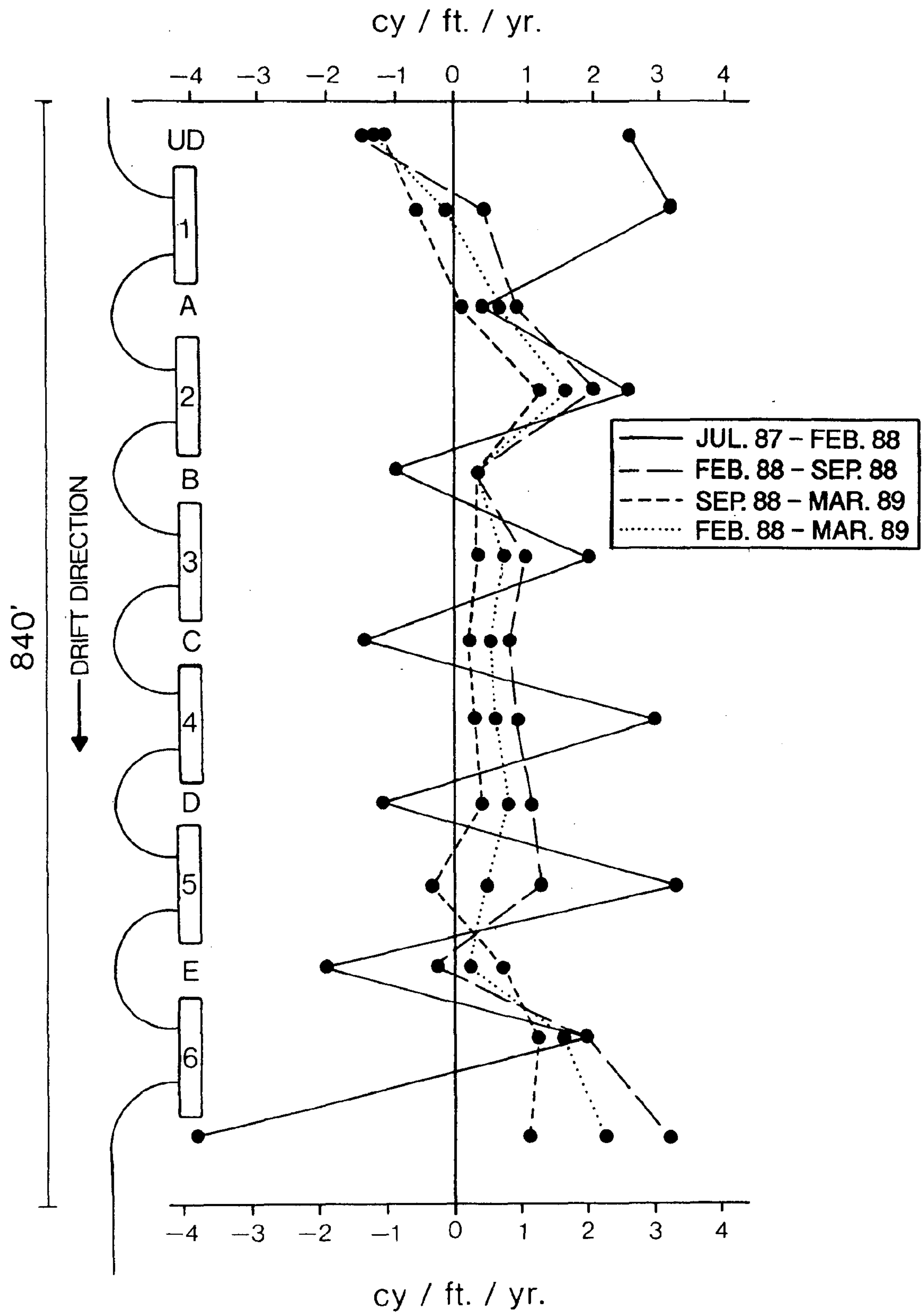


Figure 15. Chippokes State Park - Beach Volume Change.

Parkway Breakwaters, York River, York County

The Parkway breakwater system is located along the Colonial Parkway in the Colonial National Historical Park. The site is located between Sandy Point and the piers at Yorktown Naval Weapons Station (Figure 16). The historical erosion rate along this reach is approximately 1.5 ft/yr (Byrne and Anderson, 1978).

The Parkway breakwater system is situated within a shallow crenulate shaped embayment between two low headlands. The upriver headland is the terminal end of a rock revetment with a salt marsh fringe in its lee. There is a narrow salt marsh fringe approximately 800 feet downriver comprising the next headland. Prior to construction, a narrow curvilinear beach connected the two headlands. The beach is medium to coarse sand and gravel with abundant shell fragments. This material is derived from erosion of the adjacent low bank. The northerly facing tangential shore indicated a net downstream littoral drift.

Wave Climate

The shore at Parkway breakwaters faces approximately northeast and has an average fetch of 1.78 nautical miles. Changes in wave height across the York River are shown in Figure 17 for two wind velocities and water levels. Predicted wave height for a moderate northeaster is about 1.8 feet.

Design and Construction

The Parkway breakwater system was designed around the available material and construction force. The original design called for a trapazoidal cross-section similar to the rubble mound structures at Chippokes.

The Parkway breakwaters were built in May 1985. Four hundred pound concrete blocks were placed with a crane in a rectangular crib configuration. Then, concrete slabs were broken up and placed inside the crib. The cross section of each unit resembled a rectangle rather than a trapazoid.

Five units were placed at approximately the MLW line. Limitations in the equipment prevented the breakwater system from being constructed as designed (which was five equal length and equally spaced units placed at -0.5 ft MLW). As finally constructed there were five breakwater units with decreasing gap from upriver to downriver (Figure 18) (MHW = mean high water; BOB = base of bank; TOB = top of bank). It has provided a breakwater system with variable parameters for study (Table 5).

Shore Changes

The Parkway breakwaters were exposed to the no-name storm of November 4, 1985. The main direction of wave attack during the storm was east northeast with a storm surge of over 2 feet MHW. An average of 10 feet per linear foot of fastland bank was eroded. Bank erosion provided additional sand that widened the backshore. The storm mostly affected the shore between breakwaters 1, 2, and 3 and was responsible for high annual erosion rates (Table 6).

The shore at Parkway breakwaters is typically beset by frequent northwest and northerly winter winds. The relatively wide gaps and oblique incident waves have resulted in the formation of shallow crenulate bays. These gaps are most pronounced in bays A and B. The orientation of the tangential shore indicates onshore wave approach to be approximately N 25° E. Bays C and D are more narrowly spaced and are more symmetrical than bays A and B (Figure 19A).

The April 1988 northeaster caused additional bank erosion at Parkway breakwaters which also widened the backshore along most of the site. Storm waves approached the site during high tide from between 35 and 40 degrees (TN). This caused a shift in the beach planform to a more symmetrical shape with flattened embayments (Figure 19B). This general symmetrical planform persisted until the late fall of 1988 when the return of northwesterly winds reshaped the beaches into a log-spiral configuration. The late winter storms of 1989 once again shifted the embayed beach sands into a symmetrical planform under the influence of more northeasterly winds. The effects on maximum indentation in bay B are seen in Figures 20A and 21A.

Figure 22 shows net changes in beach volume. The bank and backshore are characterized by shelly, medium sand. Silty fine sands reside offshore and gravelly medium sands dominate the beach face. Beach samples show little change through time (Figure 23). Post-storm samples are not shown.

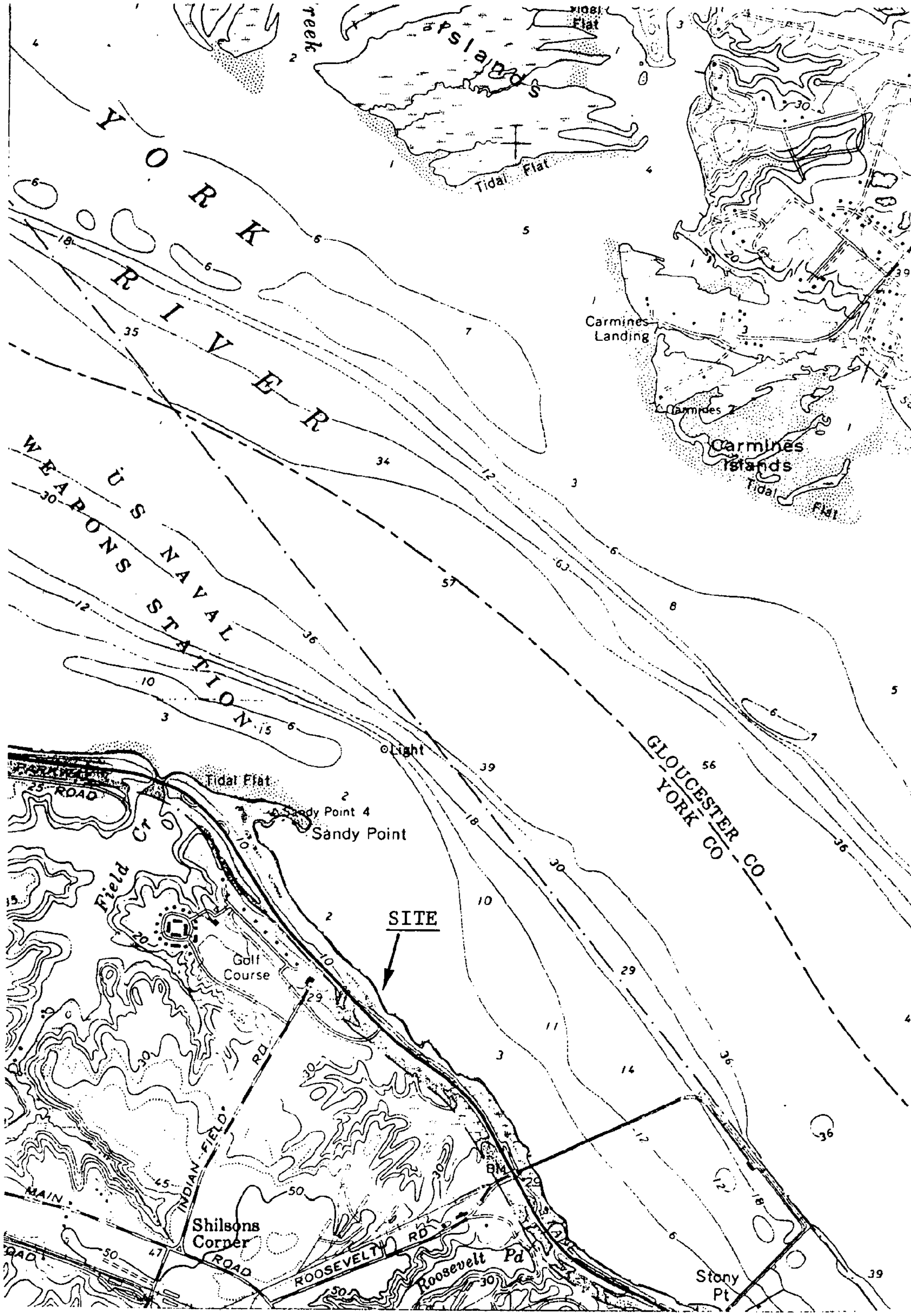
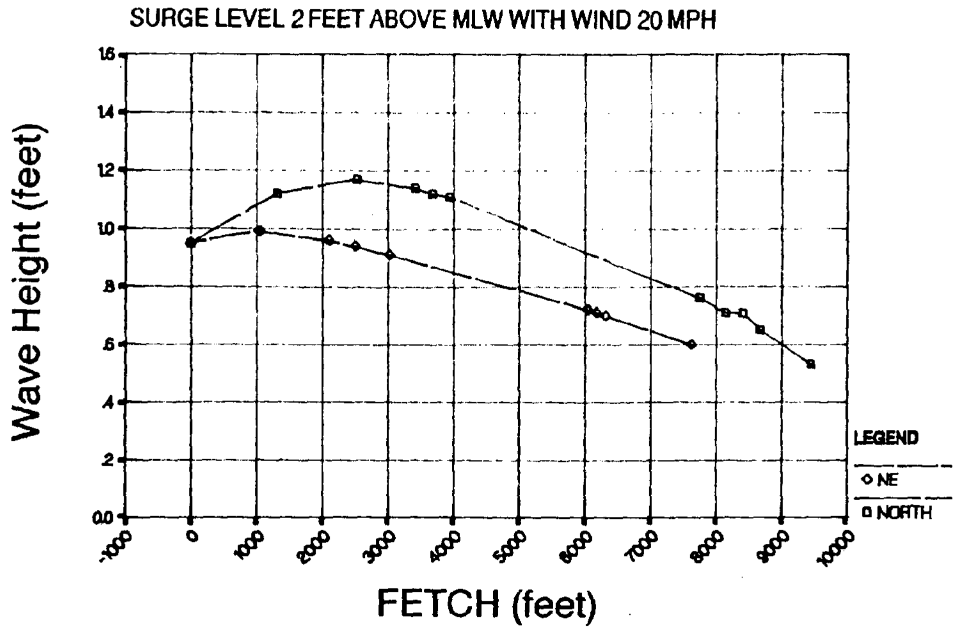


Figure 16. Parkway Breakwaters, York River, York County.
From Clay Bank 7.5 minute quadrangle.
Scale: 1 inch = 2,000 feet.



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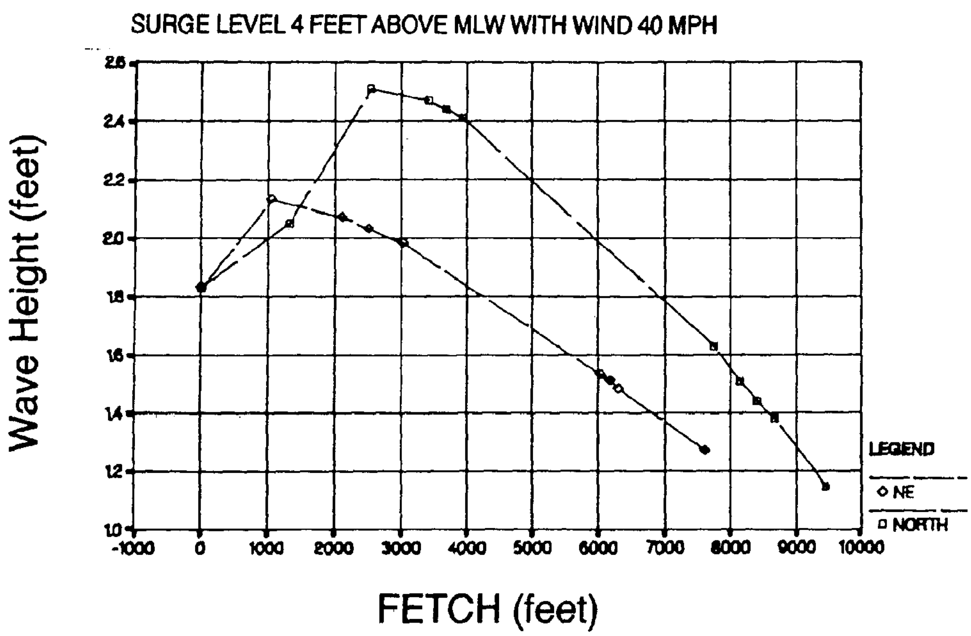


B

Figure 17. Parkway Breakwaters - Wave Climate.

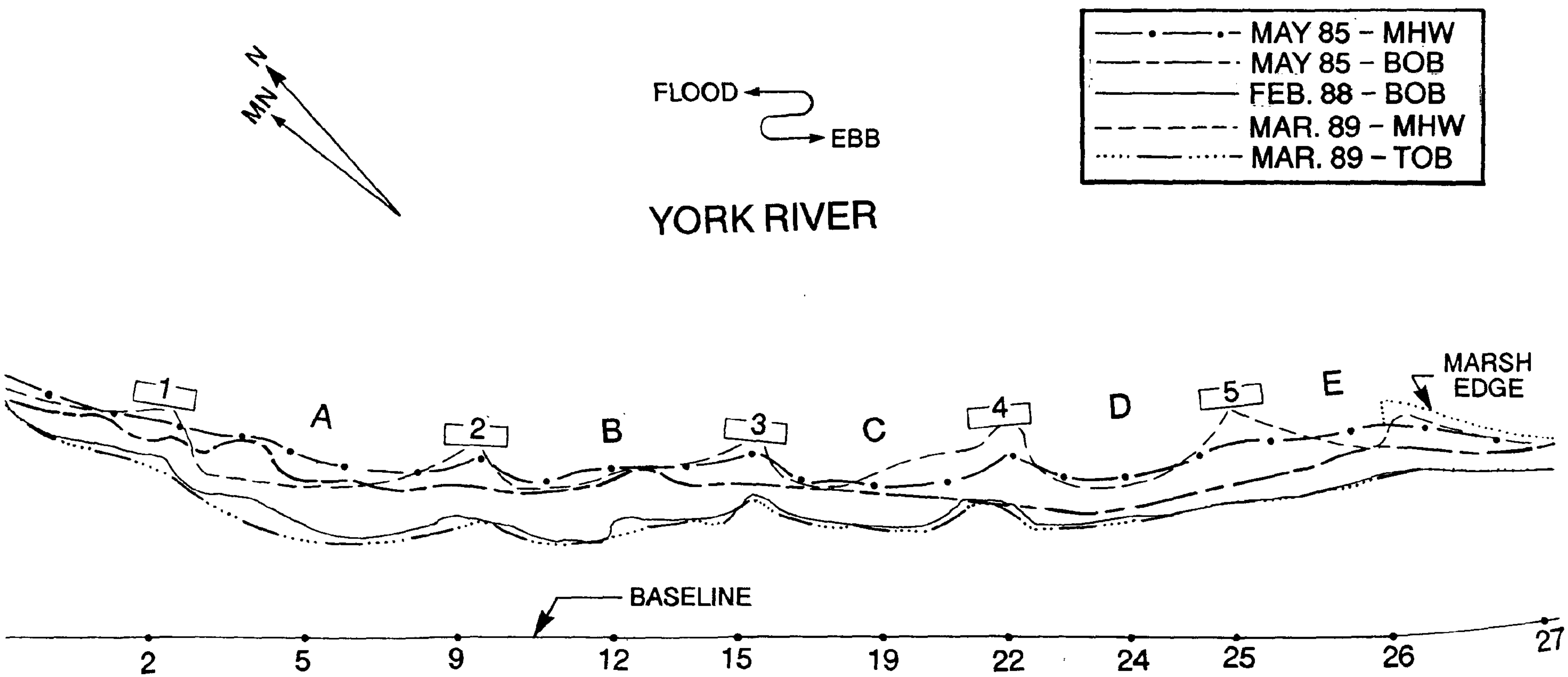


Figure 18. Parkway Breakwaters - Base Map.

Table 5. Parameters for Parkway Breakwaters*
March 1989

Breakwater/Bay	L_B	G_B	X_B	h_B	F_B	M_b	T_e	S_e	B_I	B_M**	T_A	T_W
Breakwater 1	36		14	1.9	1.3		0.4	1.0	8	25		10
Bay A		118				35		1.2	8	35		
Breakwater 2	38		10	1.9	1.3		1.1	2.6	10	35		15
Bay B		100				25		1.8	7	23		
Breakwater 3	34		10	2.3	1.3		0.7	0.8	14	26		18
Bay C		85				22		1.1	4	25		
Breakwater 4	33		20	2.6	0.5		-0.2	0.8	20	27	15	
Bay D		80				37		1.2	15	15		
Breakwater 5	22		20	2.5	0.5		0.3	1.4	20	45	5	

* All dimensions in feet.

** Distance between MHW + base of bank does not reflect change in position of the base of bank, refer to Fig. 14.

L_B - Breakwater crest length

G_B - Breakwater gap

X_B - Distance offshore CL breakwater to original MHW

h_B - Height of breakwater from bottom at CL to MHW

F_B - Breakwater freeboard, MHW to crest

M_b - Maximum bay indentation, CL breakwater to MHW

T_e - Tombolo elevation in lee of breakwater + MHW

S_e - Backshore elevation at base of bank

B_I - Initial beach width, base of bank to MHW

B_m - Present beach width, base of bank to MHW

T_A - For unattached tombolo, MHW to CL of breakwater

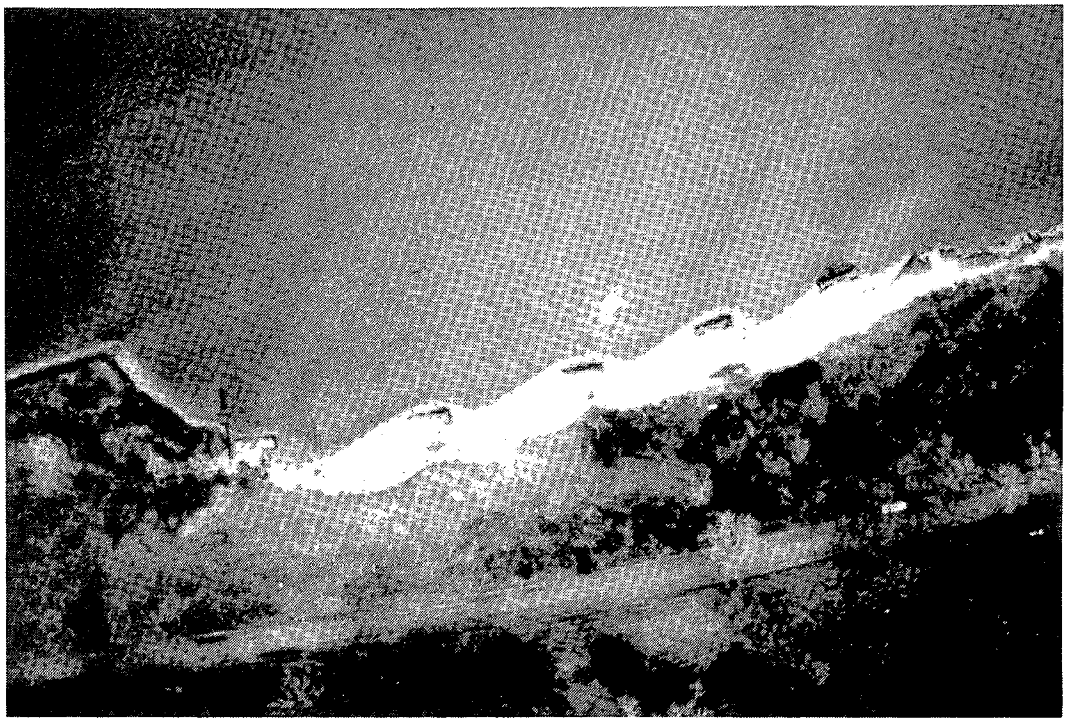
T_W - For attached tombolo, tombolo width at MHW

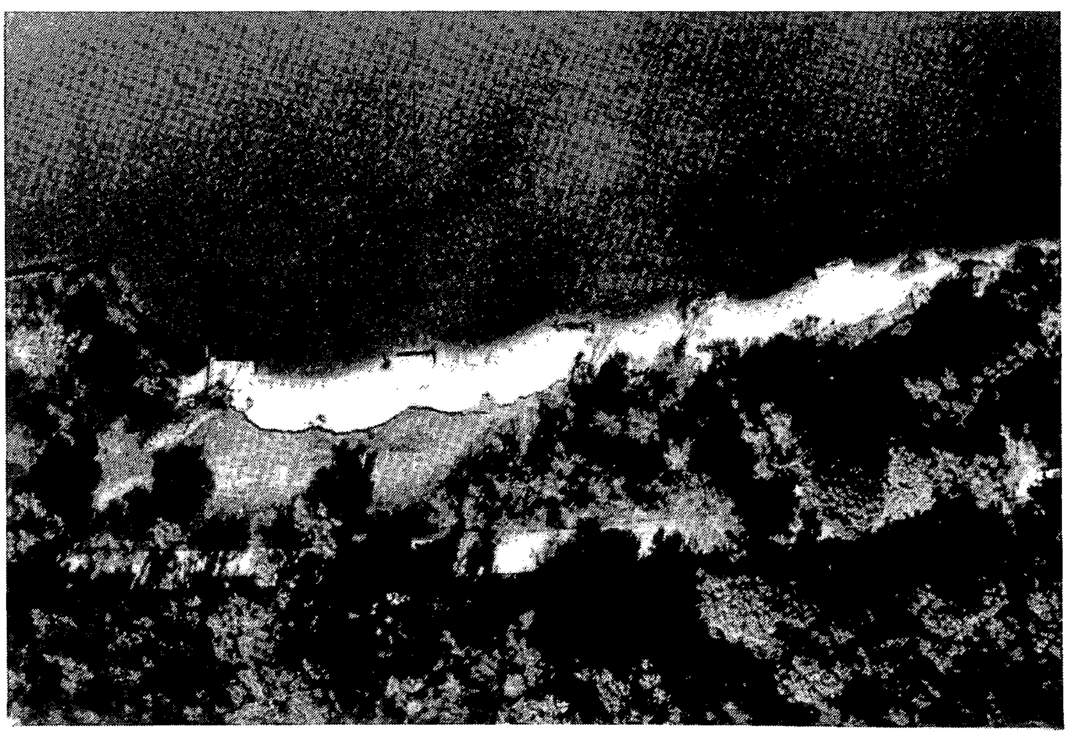
Table 6. Bank Erosion Rates, Parkway Breakwaters

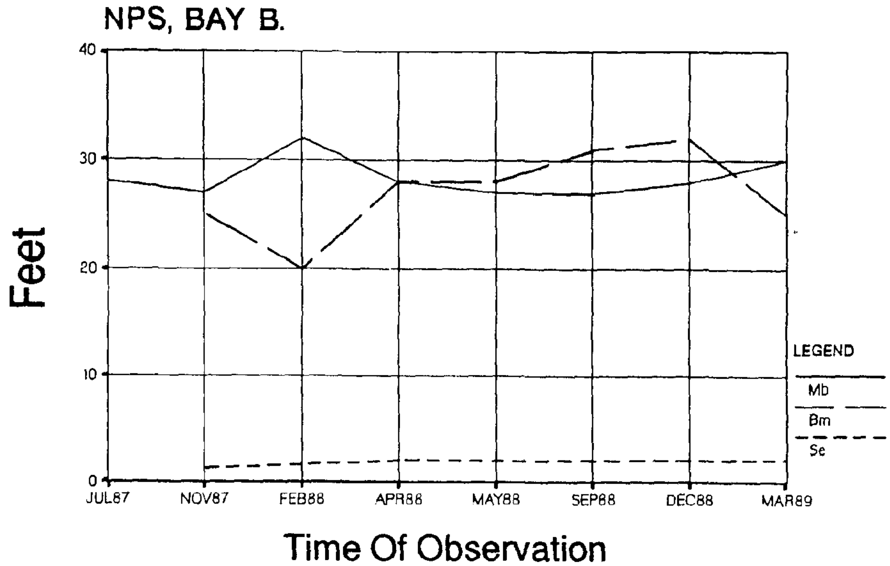
Breakwater 1	7 ft	= - 2.5 ft/yr
Bay A	23 ft	= - 8.1 ft/yr
Breakwater 2	35 ft	= -12.3 ft/yr
Bay B	20 ft	= - 7.0 ft/yr
Breakwater 3	25 ft	= - 8.8 ft/yr
Bay C	15 ft	= - 5.3 ft/yr
Breakwater 4	6 ft	= - 2.1 ft/yr
Bay D	5 ft	= - 1.8 ft/yr
Breakwater 5	10 ft	= - 3.5 ft/yr
Downdrift	20 ft	= + 7.0 ft/yr

Figure 19A. Parkway Breakwaters - aerial vertical. March 9, 1988.

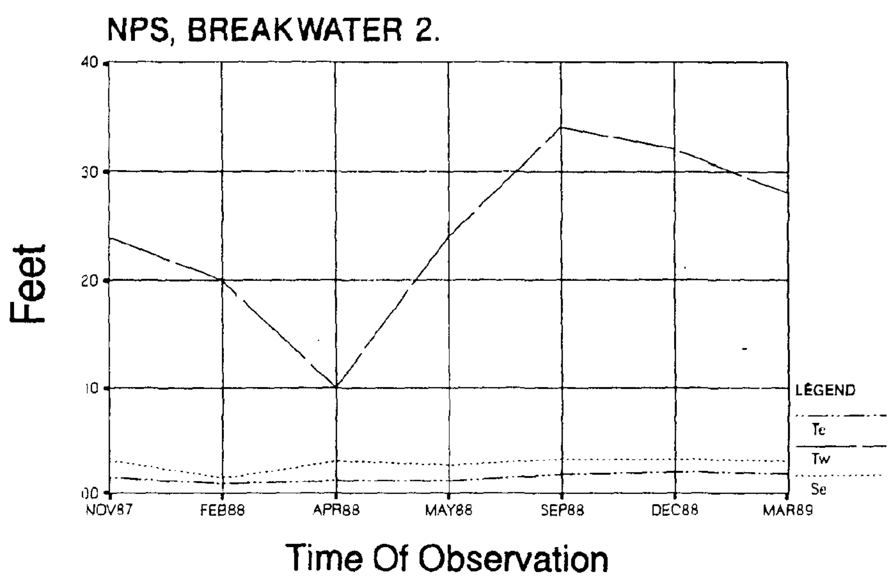
Figure 19B. Parkway Breakwaters - aerial vertical. April 20, 1988.







A



B

Figure 20. Parkway Breakwaters - Representative Parameters.

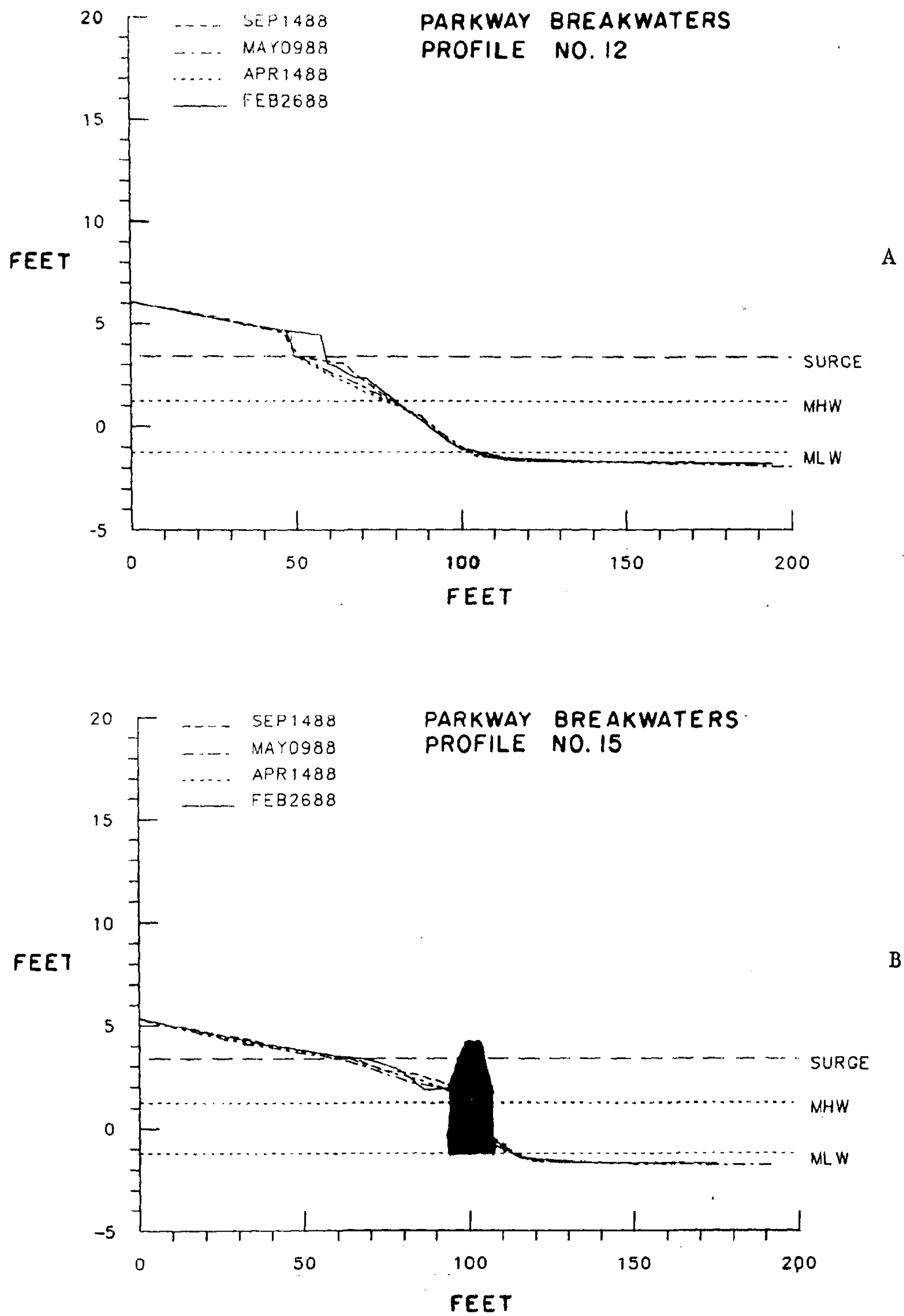


Figure 21. Parkway Breakwaters - Representative Profiles.

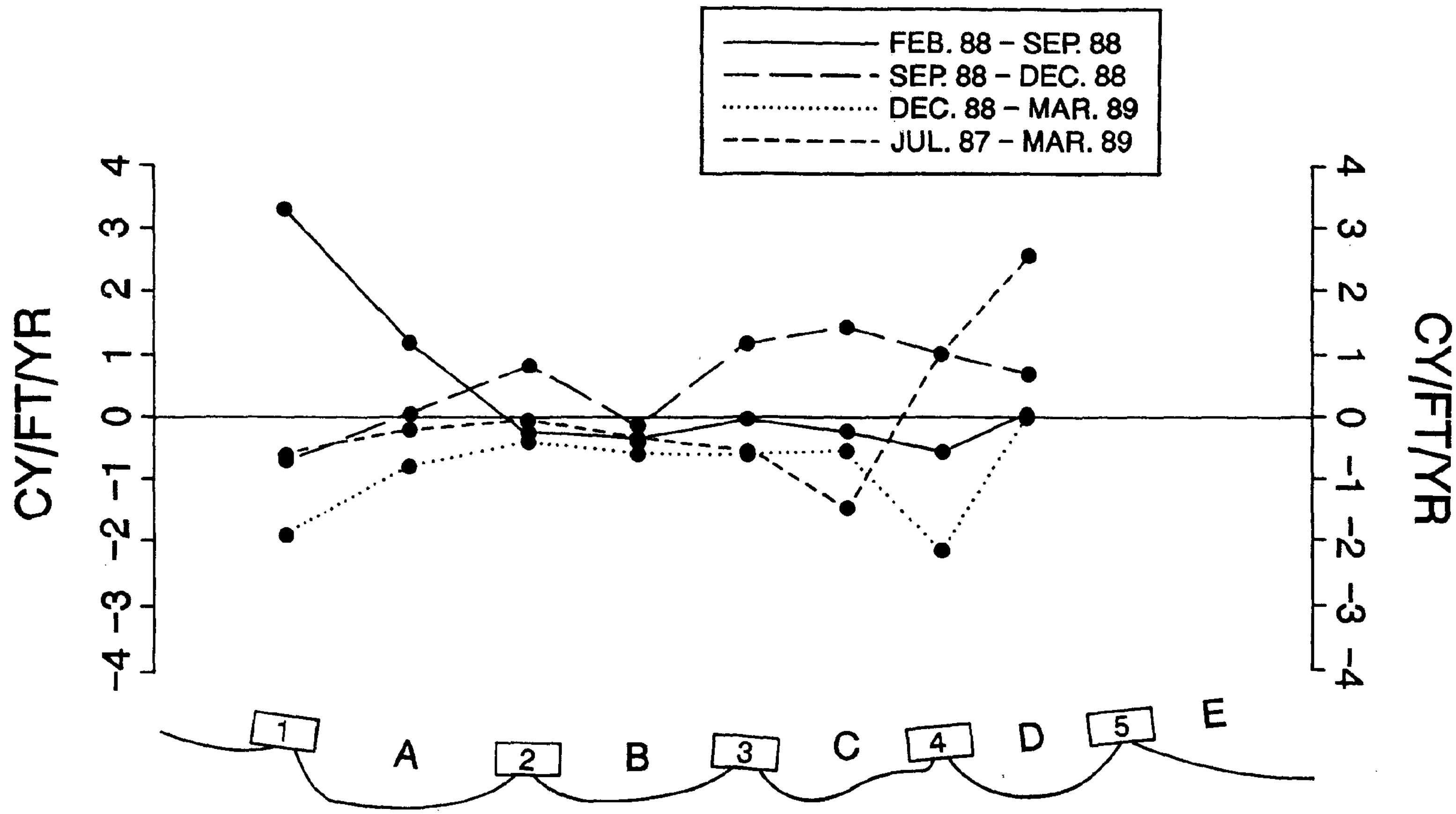
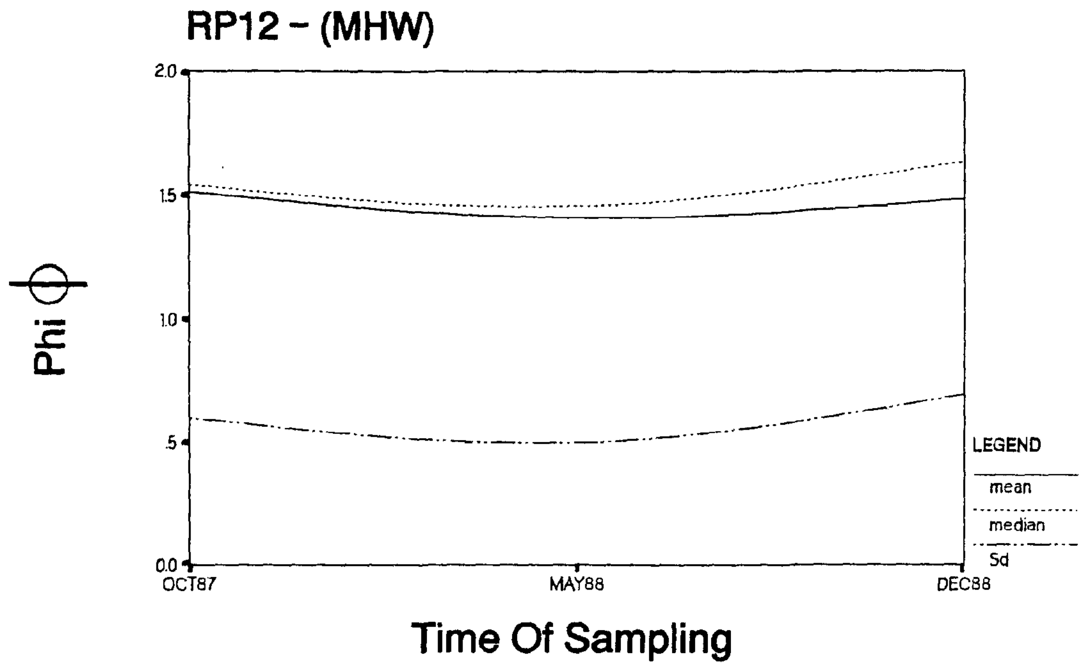


Figure 22. Parkway Breakwaters - Beach Volume Change.



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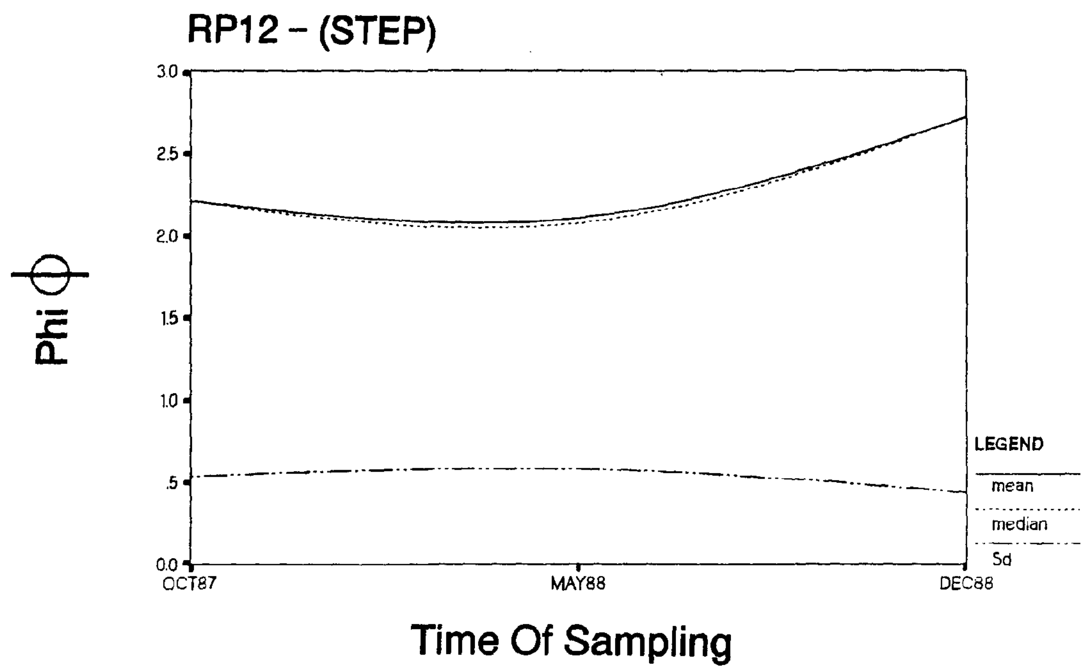


B

Figure 23. Parkway Breakwaters - Representative Beach Sediment Analysis.

Hog Island Breakwaters, James River, Surry County

The Hog Island breakwater system is located on the western shore of Hog Island on the James River (Figure 24). The site is located on Virginia's Department of Game and Inland Fisheries's Hog Island State Wildlife Management Area. The reach encompassing the Hog Island breakwaters extends from a small headland approximately 0.5 nautical miles north of Virginia Power's Surry Nuclear Power Plant's outfall northward approximately 0.7 nautical miles to a marsh headland. The baseline at the site is 1,475 feet long. The shore at the site faces west northwest with an average fetch of 2.7 nautical miles. The fastland bank is 3 to 11 feet above MSL (mean sea level) and consists of mixed clayey sands and gravels in an earthen dike composed of dredged material from nearby channels. The access road around Hog Island was built on this dike. Management of the dike allows occasional flooding of the interior of the Hog Island Wildlife Management Area.

The historical erosion rate along this reach is approximately 1.7 feet per year (Byrne and Anderson, 1978). A gabion revetment and a single groin were built along this shore in 1962. Those structures have abated the erosion of the fastland bank at this site. However, the adjacent banks have continued to recede. The gabion revetment had become a small headland by 1982.

The gabion headland resulted in two types of shoreline configuration. To the south of the structure, there was a narrow beach (3 to 5 ft from the base of the bank to the MHW line) fronting the steep, wave-cut bank of the dredged disposal dike. The northern shoreline segment had a narrow beach fronting a low, wooded terrace which runs along the dike for 700 feet before the dike becomes close to the water again. Net littoral sediment transport to the south is indicated by the impoundment of sand and wider beaches along the northern segment. An associated decrease in beach width is observed for several hundred feet south of the gabion headland.

Wave Climate

The average fetch at Hog Island breakwaters is approximately 2.7 nautical miles westward up the James River. After a northeast storm, the winds often shift quickly to the northwest and the storm surge will remain for a few hours. It is during this post-storm period that wave action will significantly affect westerly-facing shores such as Hog Island breakwaters (Figure 25B).

Design and Construction

The Hog Island breakwaters were installed in June 1987. The purpose of this project is to examine the effects of breakwaters of varying lengths, heights, and offshore distances. The use of salt marsh grass implantation for shore erosion control had been tested here in 1982 and 1983 without success. The grasses washed out during the winter storm seasons in both 1983 and 1984 (Hardaway et al., 1984).

Six pairs of breakwaters were built using two construction methods. The first method was installation of rubble mound breakwater units using a backhoe. The second method employed the use of gabion baskets, placed by hand, to hold rocks ranging from 50 to 150 pounds, each loaded by front-end loader.

Six breakwater units were placed south of the gabion headland and six breakwater units were placed north (Figure 26). Breakwater units 1, 2, 5, 6, 9 and 10 were constructed of gabions. The remaining breakwaters (3, 4, 7, 8, 11 and 12) were constructed as rubble mounds.

Approximately 1,000 cubic yards of beach fill was placed along the northern section and 500 cubic yards on the southern section. The mean diameter of the fill was 0.4 mm (1.23 phi). The fill was truck hauled from a borrow pit near Smithfield, Virginia. After installation of the breakwaters, the higher fastland banks were graded.

Shore Changes

The sand fill increased the beach width several feet. Immediately after installation, a cuspate spit began to form behind each breakwater unit. In general, this occurred by wave action taking sand from the embayments.

By March 1988, the position of the MHW line behind breakwater units 1, 2, 3, 4, 7 and 8 had receded and the MHW line behind breakwater units 5, 6, 9 and 10 had stabilized. The cuspate spits which fully attached (i.e. beach elevation above MHW) to become tombolos were behind breakwater units 11 and 12 (Figure 26). Breakwater units 11 and 12 are higher and wider than the other structures and are more capable of holding and maintaining a tombolo. The general trend is for the cuspate spits and/or tombolo to become wider and higher as the breakwater becomes wider, higher and longer (Table 7).

The April 1988 northeaster produced moderate wave action at Hog Island breakwaters, mostly high water from the associated storm surge (Figure 27B). The effect on the beach was to deflate the embayments along the entire site (Figure 28B, D). The low crested breakwater tombolos were also reduced, but the high crested breakwaters gained tombolo elevation (Figures 28A, C). The principal wave action from the strong northwesters during the fall and winter usually affects only the intertidal beach. The upland banks erode during post-storm conditions when winds shift from northeast to northwest on top of the storm surge.

The effect on bay beach parameters through time is seen in Figures 29A, B, C and D. Beach widths (Bm) are restricted in the two bay types due to close proximity to the upland bank. The trend shows as bay depth (Mb) decreases, beach width (Bm) increases for both bay types. Bay E showed a noticeable decrease in backshore elevation (Se) due to the April 1988 storm, whereas Se in bay L remained relatively stable. Backshore elevation (Se) behind each type breakwater shows an unexpected trend and becomes smaller as tombolo elevation (Te) becomes larger. This may be caused by offshore movement of the beach behind each structure

Net volume changes along the entire site show a slight accretion from breakwater 1 to 5 and slight sediment loss across the remainder of the system from June 1987 to March 1989 (Figure 30).

The beach fill material at the Hog Island breakwaters, shortly after being emplaced, was characterized as slightly silty, gravelly sands. Figure 31 shows the change in beach material size and sorting through time for two bay types at high water and at the beach step.

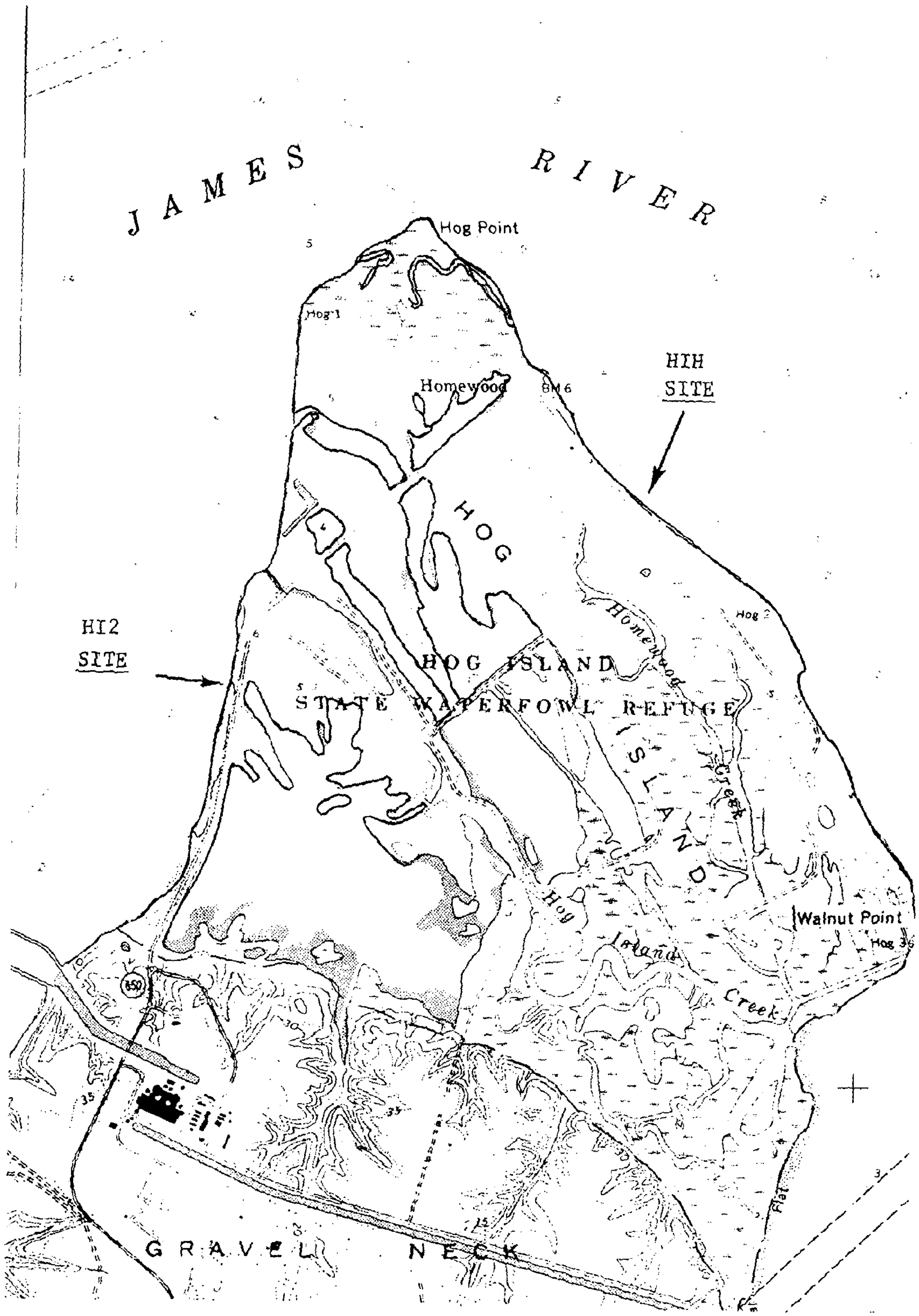
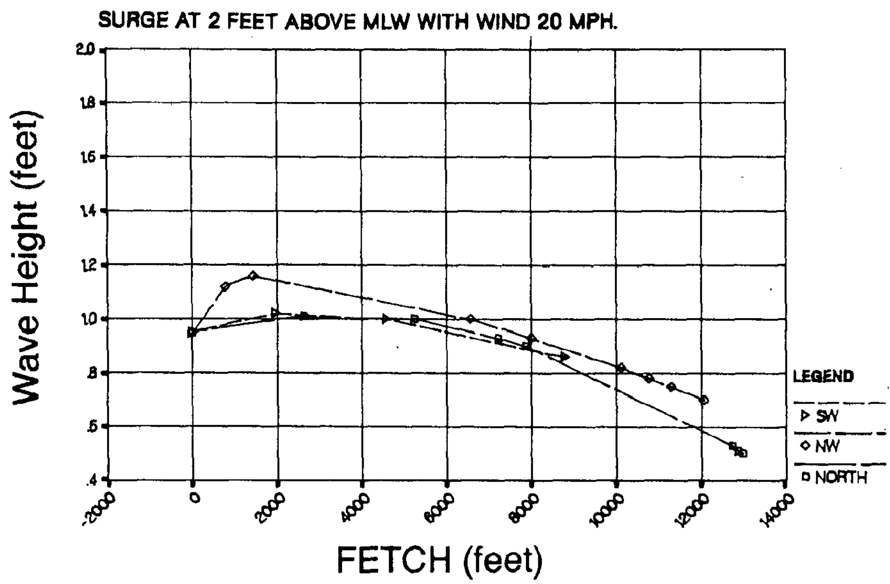


Figure 24. Hog Island Breakwaters and Headlands, James River, Surry County.
From Hog Island 7.5 minute quadrangle.
Scale: 1 inch = 2,000 feet.



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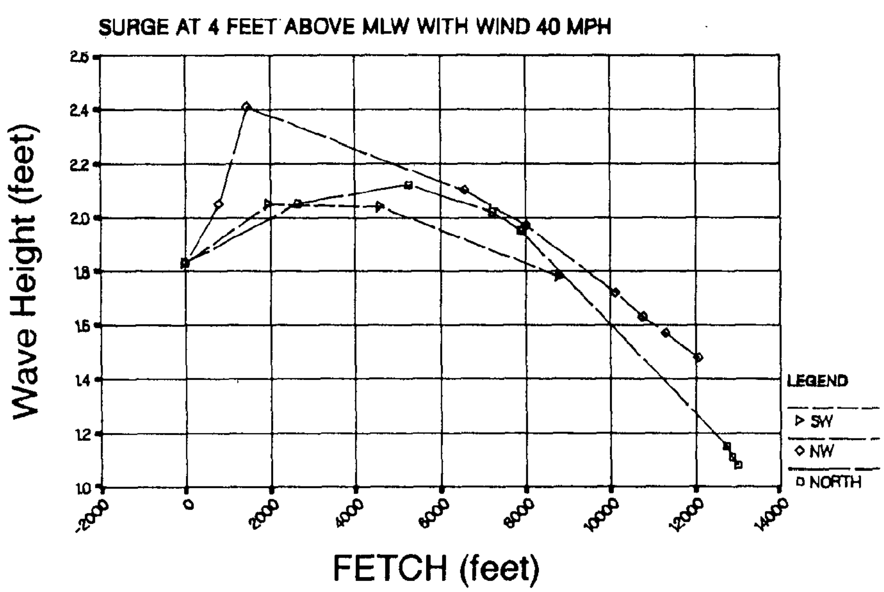


B

Figure 25. Hog Island Breakwaters - Wave Climate.

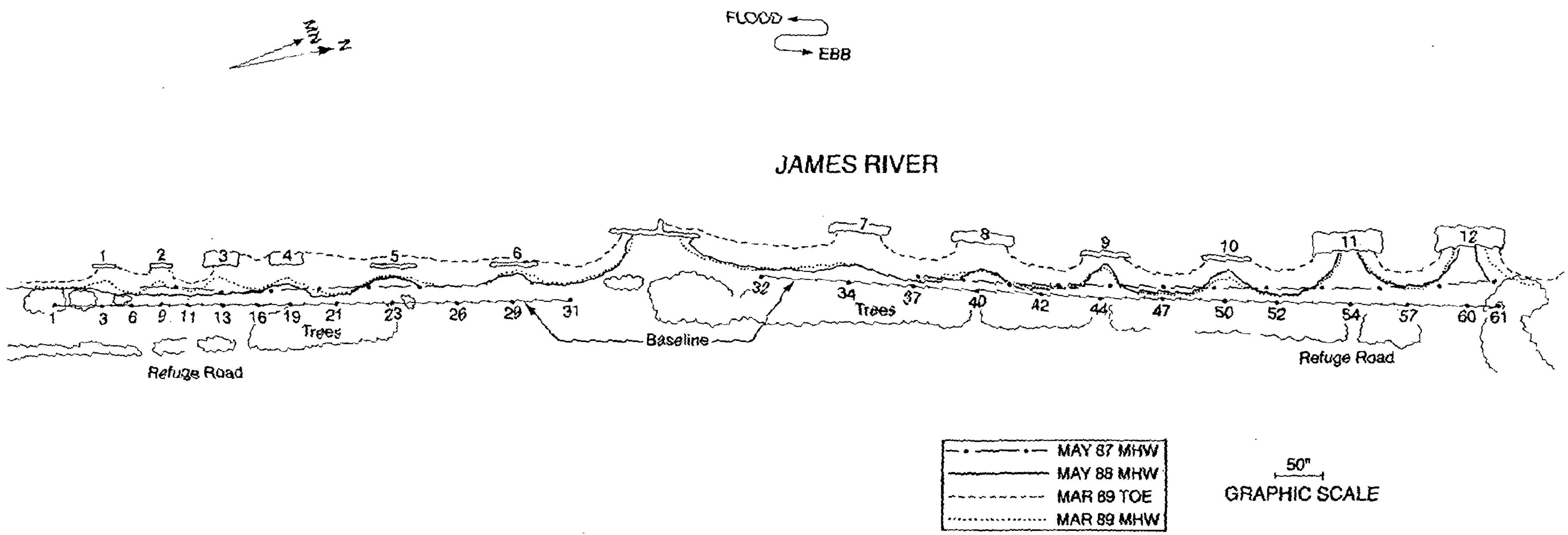


Figure 26. Hog Island Breakwaters - Base Map.

Table 7. Parameters for Hog Island Breakwaters*
March 1989

Breakwater/Bay	L_B	G_B	X_B	h_B	F_B	M_b	T_e	S_e	B_I	B_M	T_A	T_w
Updrift								0.5	5	1		
Breakwater 1	24		17	2.0	1.0		−0.8	1.0	4	17	9	
Bay A		36				21		0.8	5	4		
Breakwater 2	24		21	2.4	0.6		−0.7	1.1	4	12	13	
Bay B		36				27		0.7	5	5		
Breakwater 3	25		32	3.5	0.5		−0.7	1.2	1	21	16	
Bay C		42				27		1.2	2	12		
Breakwater 4	25		32	3.5	0.5		−0.9	0.3	2	8	21	
Bay D		72				29		1.0	7	9		
Breakwater 5	48		20	2.2	0.8		−0.8	1.2	2	16	7	
Bay E		76				22		0.8	3	6		
Breakwater 6	48		19	1.5	1.5		−0.5	1.8	3	18	7	
Bay F		70				33		0.6	4	5		
Gabion Revetment	92											55
Bay G		136				36		1.0	5	7		
Breakwater 7	50		45	3.5	−0.9		−1.9	1.2	10	18	36	
Bay H		77				46		1.2	7	6		
Breakwater 8	50		43	3.8	−0.4		−1.6	1.3	10	20	32	
Bay I		75				43		0.8	8	5		
Breakwater 9	48		30	3.3	1.2		−1.3	1.6	7	30	13	
Bay J		75				42		0.5	15	2		
Breakwater 10	48		29	3.0	1.5		−0.9	1.7	12	20	22	
Bay K		70				42		−0.1	5	0		
Breakwater 11	50		46	3.4	1.8		0.3	2.2	17	53		19
Bay L		75				43		0.8	3	7		
Breakwater 12	50		45	3.6	1.8		0.2	2.6	5	47		25
Downdrift								1.6	5	18		

Table 7 (cont'd.)

* All dimensions in feet.

L_B - Breakwater crest length

G_B - Breakwater gap

X_B - Distance offshore CL breakwater to original MHW

h_B - Height of breakwater from bottom at CL to MHW

F_B - Breakwater freeboard, MHW to crest

M_b - Maximum bay indentation, CL breakwater to MHW

T_e - Tombolo elevation in lee of breakwater + MHW

S_e - Backshore elevation at base of bank

B_I - Initial beach width, base of bank to MHW

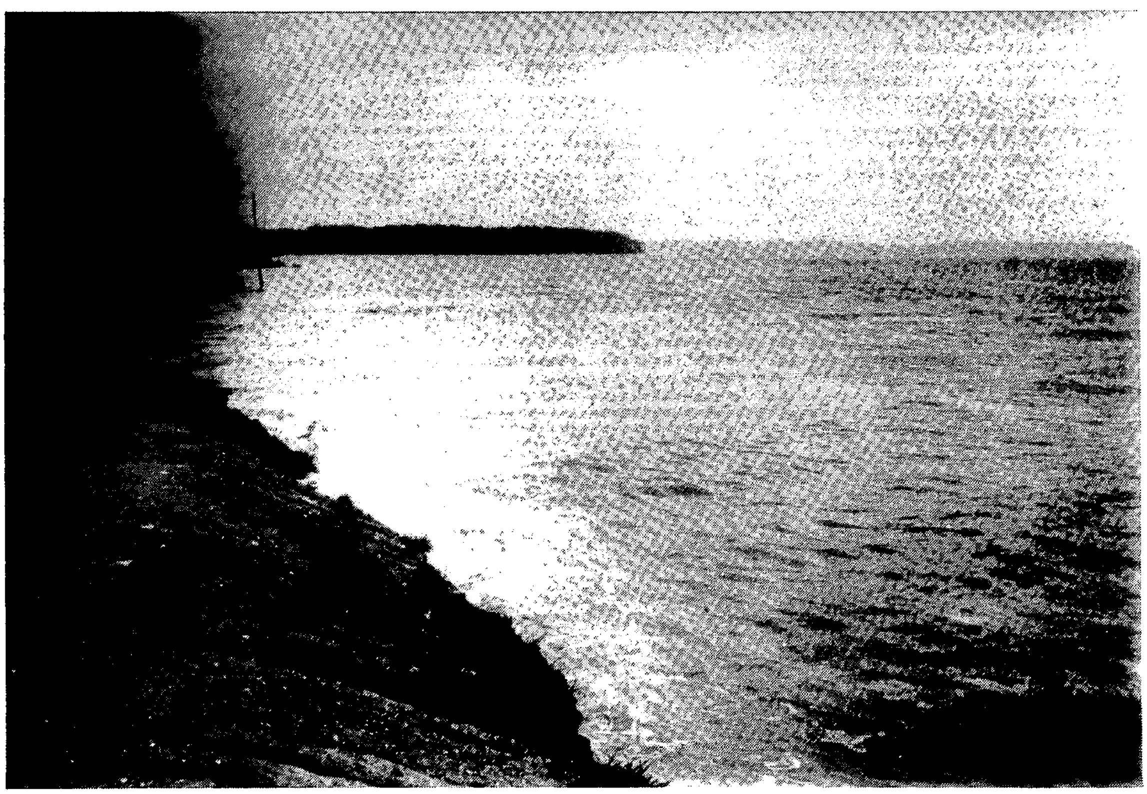
B_m - Present beach width, base of bank to MHW

T_A - For unattached tombolo, MHW to CL of breakwater

T_W - For attached tombolo, tombolo width at MHW

Figure 27A. Hog Island Breakwaters - ground view, looking south. April 15, 1988, post-storm.

Figure 27B. Hog Island Breakwaters - ground view, looking south. April 13, 1988, during northeaster. Use telephone pole along shore for reference.



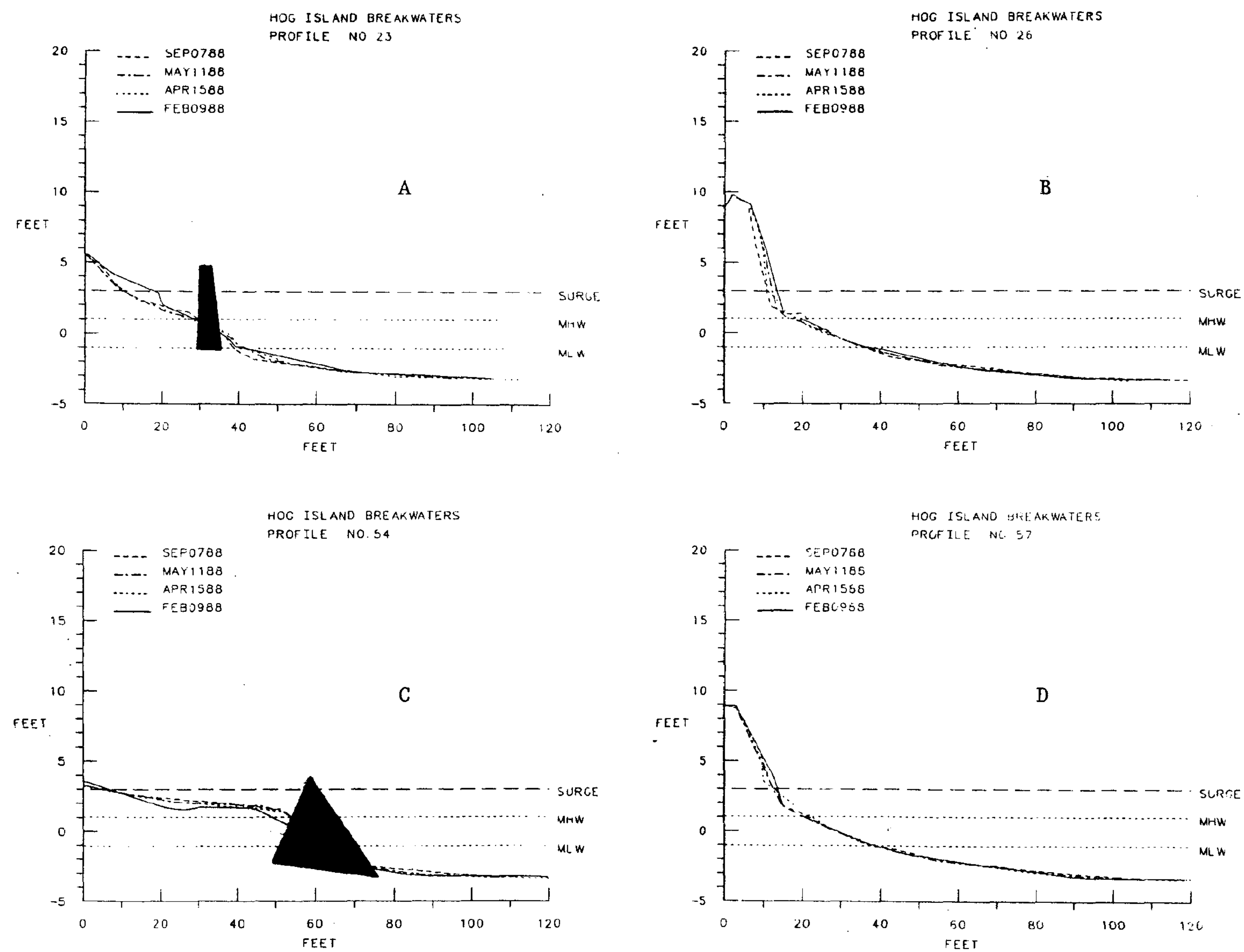


Figure 28. Hog Island Breakwaters - Representative Profiles.

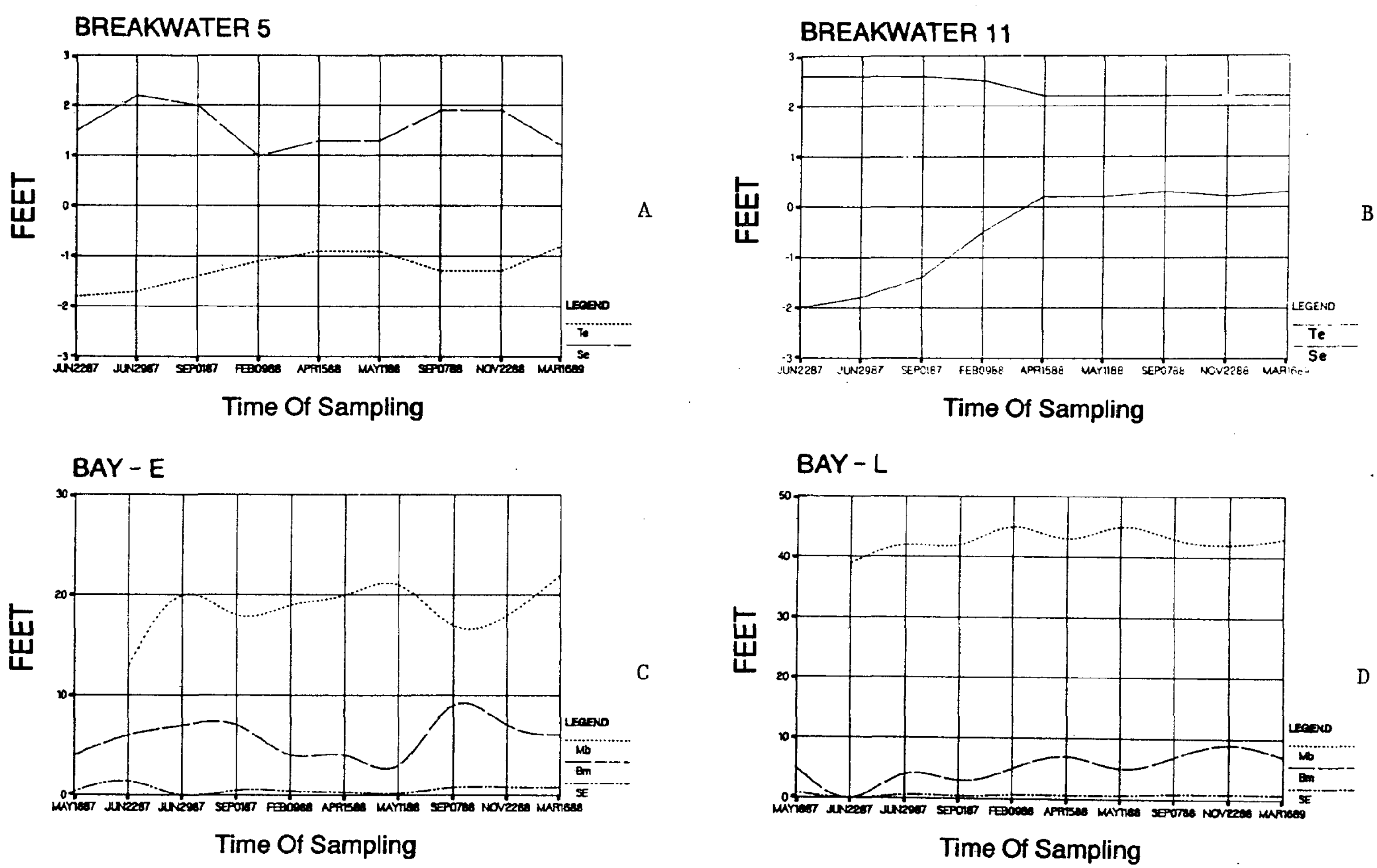


Figure 29. Hog Island Breakwaters - Representative Parameters.

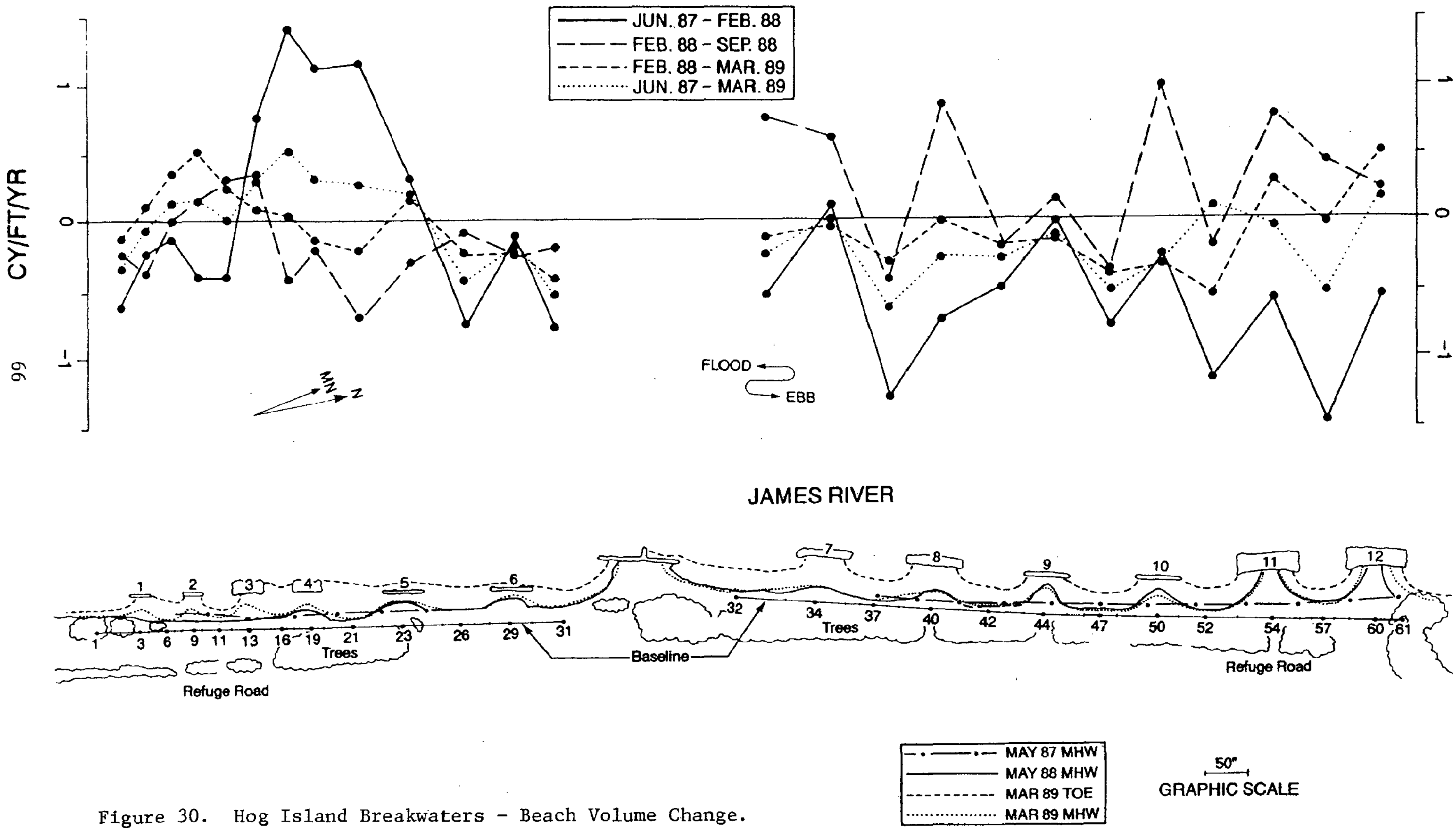


Figure 30. Hog Island Breakwaters - Beach Volume Change.

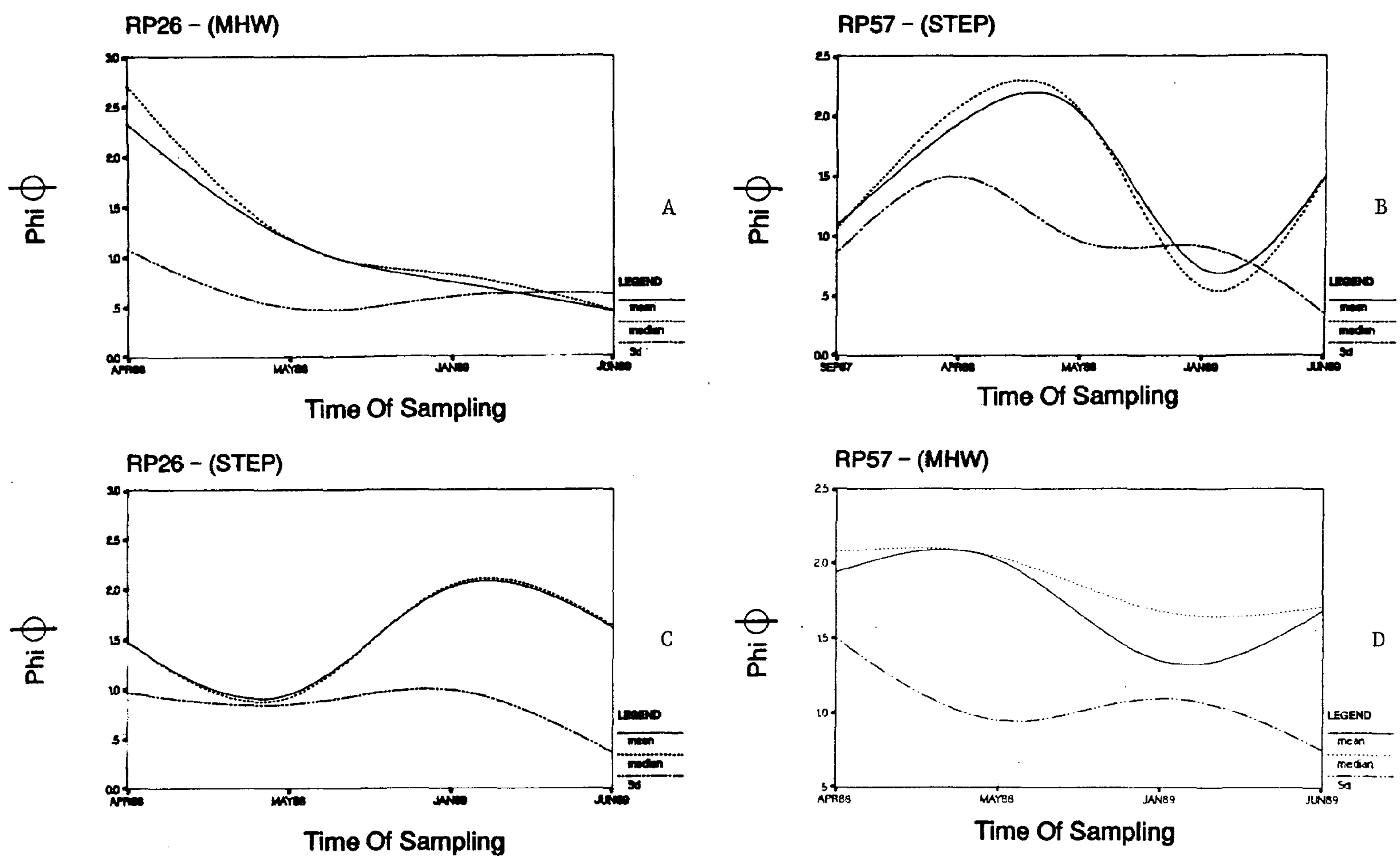


Figure 31. Hog Island Breakwaters - Representative Beach Sediment Analysis.

Drummonds Field, James River, James City County

Drummonds Field is a private development located on the north shore of the James River just west of the Jamestown Ferry's pier (Figure 32). The Drummonds Field development which was established in 1985, has approximately 1,300 feet of river frontage. Drummonds Field is situated in a reach whose boundaries are defined by the boat basin at Lake Pasbehegh to the northwest and the pier at the Jamestown Ferry to the southeast. The historic shoreline erosion rate is approximately 1.6 feet per year (Byrne and Anderson, 1978).

The 25-foot high bank at Drummonds Field is composed of a blue-grey, very stiff clay overlain by a fine to medium sand. The clay layer is an aquaclude causing intermittent springs to occur along the bank. Bank erosion provided sand to the narrow beach which, in turn, provided little or no buffer to wave action under storm conditions.

The Drummonds Field breakwater system was installed in September 1985. The purpose of the system was to provide a stable, protective and recreational beach for this waterfront development.

Wave Climate

There is an average fetch to the southwest of approximately 3.5 nautical miles. Frequent westerly winds dominate the Drummonds Field shore (Figure 33). Wave refraction analysis using 60 mph winds at +3.0 feet MLW from the west, produce an incident wave height of 3.0 feet and a wave period of 3.5 seconds. The onshore angle of wave approach is approximately 225° (TN) which is almost shore-normal.

Design and Construction

In the initial design phases, the developers wanted to protect the eroding banks and considered a bulkhead or revetment. However, such a structure would preclude any natural accumulation of beach material since it would cut off a major source of sand (i.e. the eroding fastland banks). Because of a strong desire for a recreational beach, artificial nourishment to form a defensive shoreline structure was considered. The retention of the fill was problematic and the cost estimates were high.

To accomplish the goals of a recreational beach and a protected bank, a gapped breakwater system was recommended by VIMS (developers had contacted VIMS seeking advice.) At the time in the Chesapeake Bay estuarine system, there were examples of breakwater systems at VIMS in Gloucester Point, Colonial Beach in Westmoreland County, and the aforementioned Parkway breakwaters in the Colonial National Park in York County. These were used to compare cost and effectiveness.

A field and aerial photo investigation of Drummonds Field revealed a naturally occurring crenulate bay (Figure 34A). The two headlands which defined the bay were groups of cypress trees. The trees had reduced the erosion of the fastland and allowed the adjacent banks to evolve into a rough crenulate bay with a log-spiral section and a tangential section. From the tangential section, a net angle of wave approach of approximately 225^o was determined.

An offshore breakwater was placed in front of each natural headland in order to reinforce the cypress tree headlands and, thus, stabilize the natural bay. Three more breakwaters were placed downriver at approximately the same spacing as the natural headlands and approximately 100 feet from MHW (Figure 34B). One small offshore breakwater was placed upriver. Along with these structures, over 10,000 cubic yards of beach fill was emplaced.

Shore Changes

The backshore elevation of the newly created beach was designed for protection from a storm surge of + 4.5 MSL. Construction of Drummonds Field was still in progress on September 27, 1985, when Hurricane Gloria passed offshore. The effect on Drummonds Field was a 2-foot storm surge accompanied by northwest winds of 50 to 60 mph. Wave heights were measured at 1.5 to 2.0 feet. The breakwaters were in place, and enough beach fill was present, to prevent major damage to the fastland bank.

The post beach-fill planform left the MHW line approximately 50 feet behind breakwaters 1, 2 and 3. Breakwaters 4, 5 and 6 were semi-attached. Tombolo development proceeded with partial attachment by January 1986. An additional 3,000 cubic yards were added in April of 1986. This was placed mostly behind breakwaters 1, 2 and 3 and enhanced the tombolos. Because the upriver bank continued to erode, it was evident that breakwater 6 was too short. It was extended with a dog-leg addition approximately 80 feet long (Figure 35).

By March 1988, the tombolos at breakwaters 1, 2 and 3 were firmly attached at MSL but the MHW line was still detached behind breakwaters 2 and 3 (Figure 35). Breakwaters 4, 5 and 6 were also attached at MSL with the MHW line several feet away. Also, it became necessary to abate bank erosion between breakwaters 5 and 6. Another adjustment to the system was made by adding an extension to breakwater 5 and placing several hundred cubic yards of beach fill in bay E.

Due to the southwest exposure of Drummonds Field, the April 1988 northeaster had little effect except for high water from the storm surge. However, subsequent northwesters appeared to shift material from bay D to breakwater 4 (Figure 36).

Following an initial period of adjustment, the embayed beaches have remained fairly stable in terms of backshore elevation (Se), beach width (Bm) and bay depth (Mb) (Figure 37). The status of site parameters is shown in Table 8. Net volume changes (Figure 38) are difficult to interpret due to the intermittent additions of beach fill by the residents and frequent beach grading to remove vegetation. Thus, the general shore planforms show accretionary patterns. The shapes of bays A, B, C and D are generally symmetrical, flattened and curvilinear with a slight log-spiral pocket on the upstream side.

Beach sediment samples, especially the lower beach (step), show a sharp increase in grain size after the April 1988 storm with an equally sharp decrease by December 1988 (Figure 39). The bay beaches appear to fluctuate locally but maintain a generally stable planform through time.

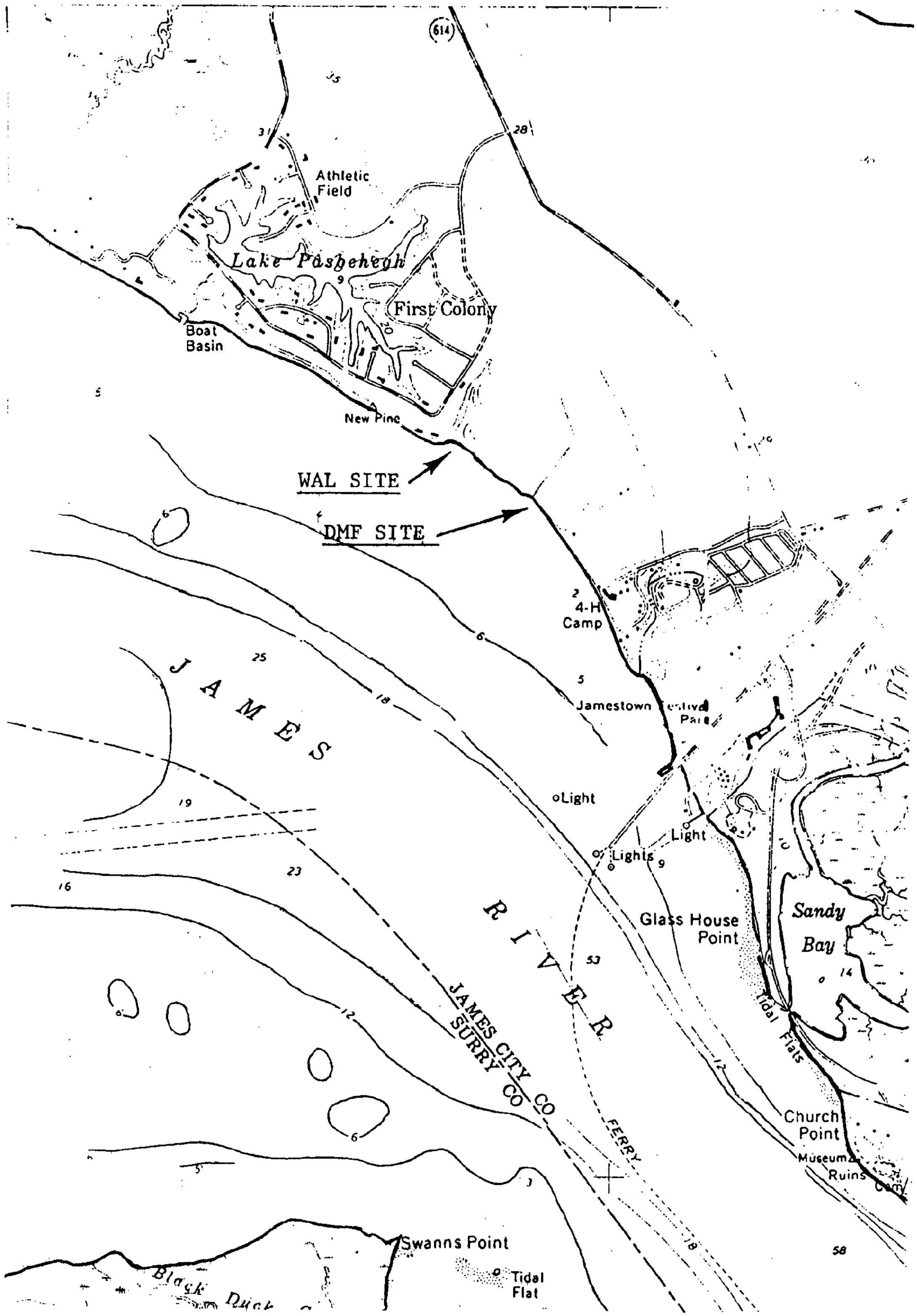


Figure 32. Drummonds Field and Waltrip, James River, James City County.
From Surry 7.5 minute quadrangle.
Scale: 1 inch = 2,000 feet.

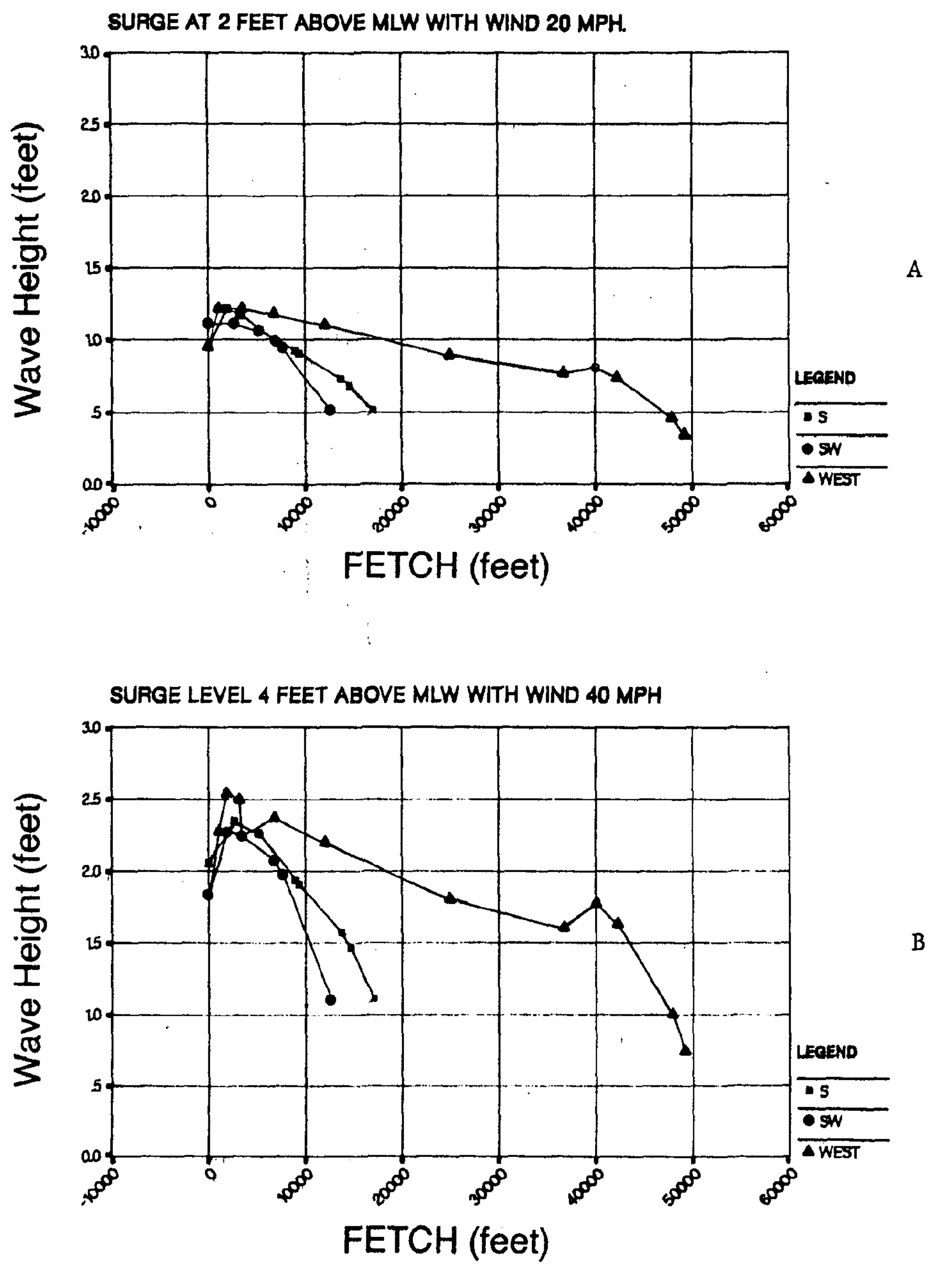
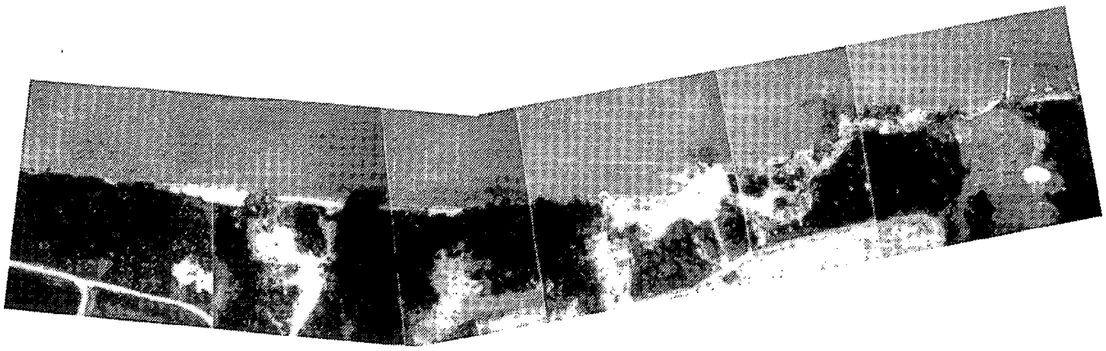
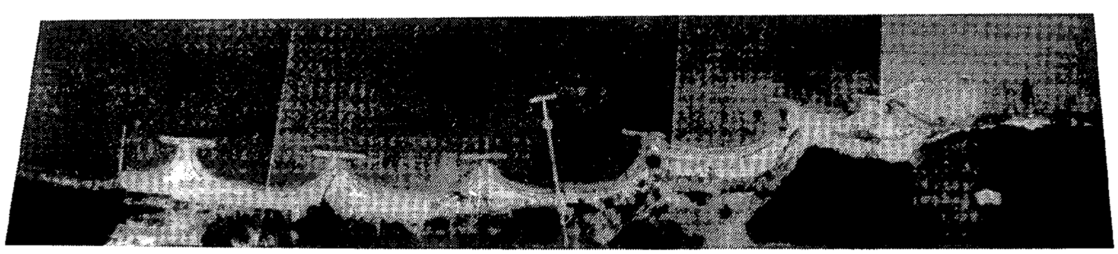


Figure 33. Drummonds Field and Waltrip - Wave Climate.

Figure 34A. Drummonds Field - aerial vertical. September 11, 1985. Pre-construction.

Figure 34B. Drummonds Field - aerial vertical. July 15, 1986. Post-construction.





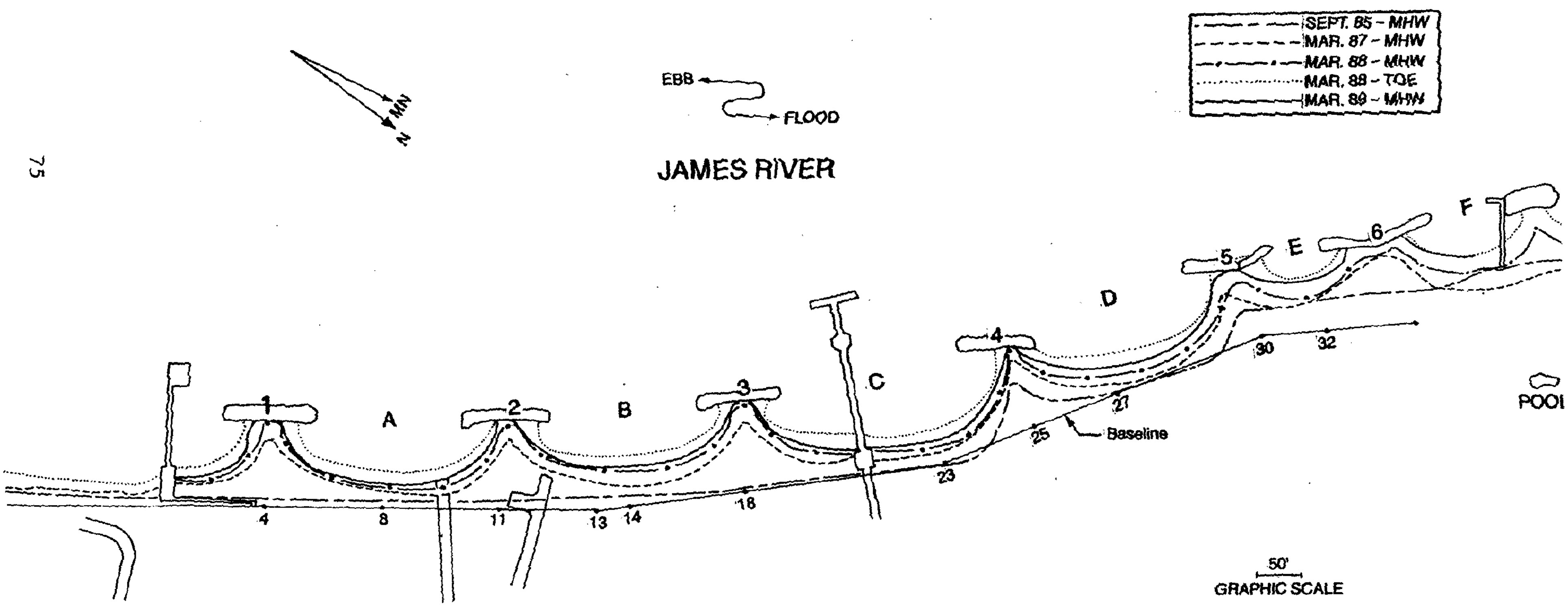
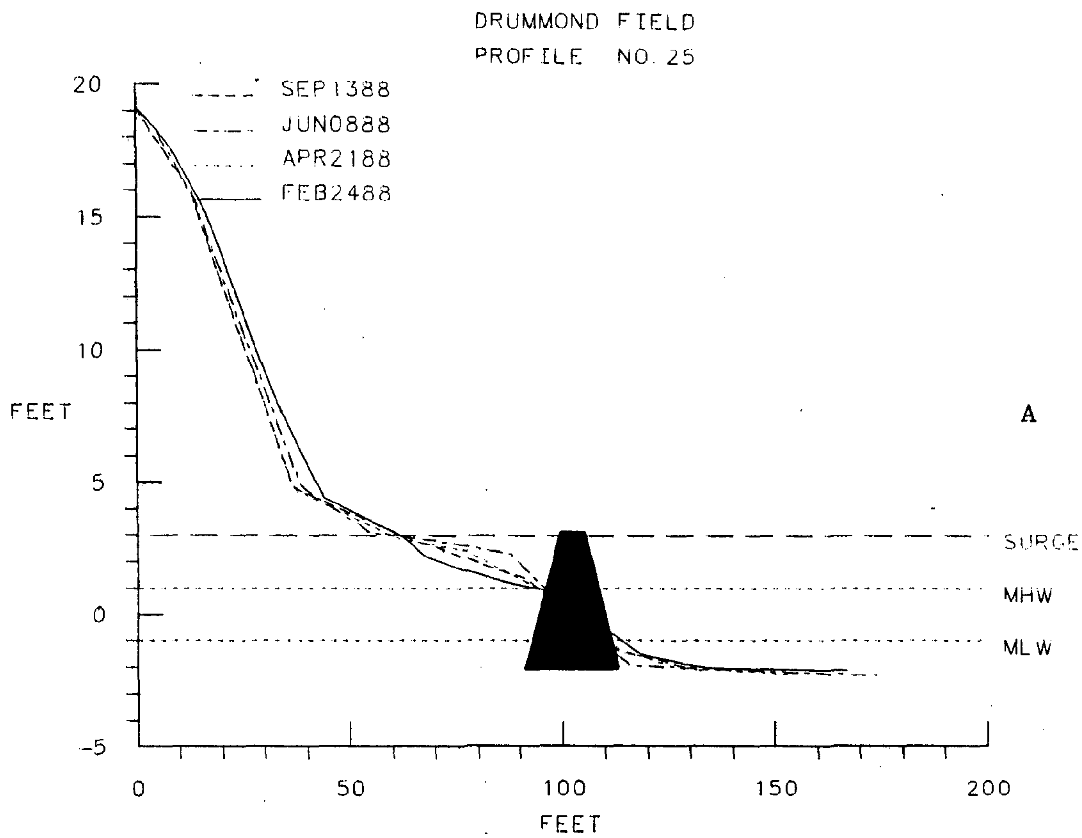


Figure 35. Drummonds Field - Base Map.



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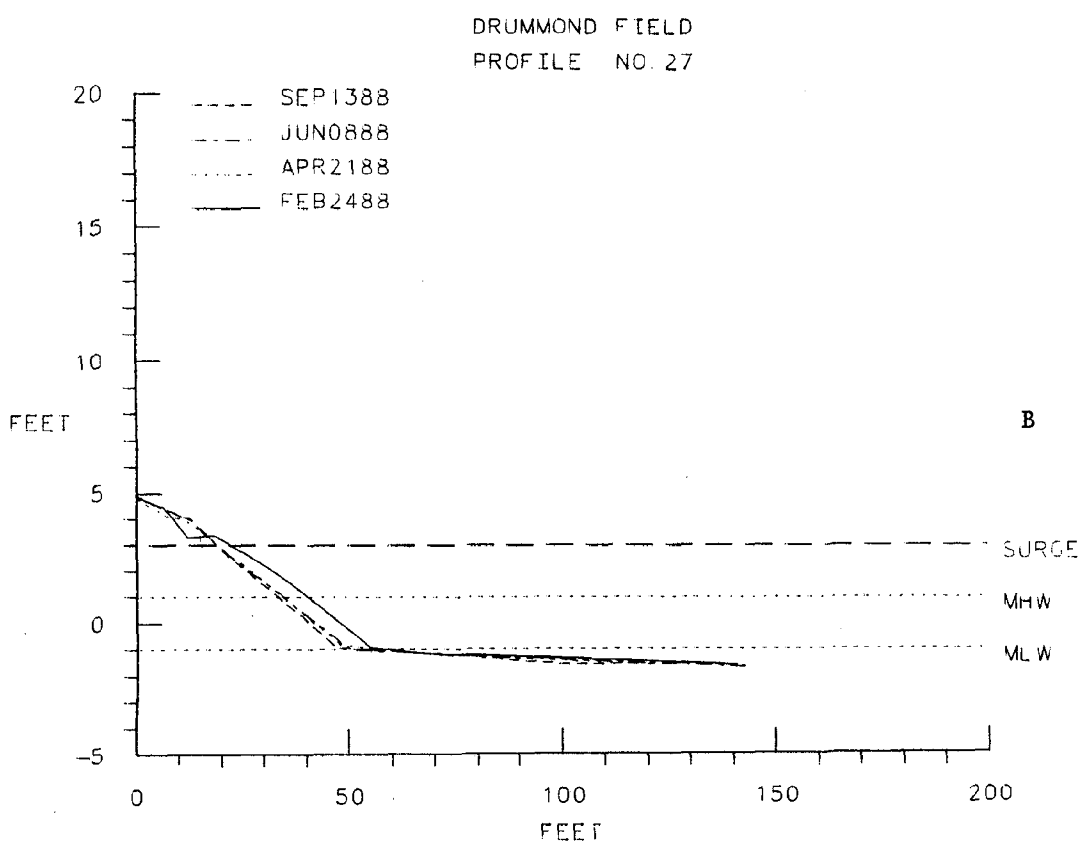
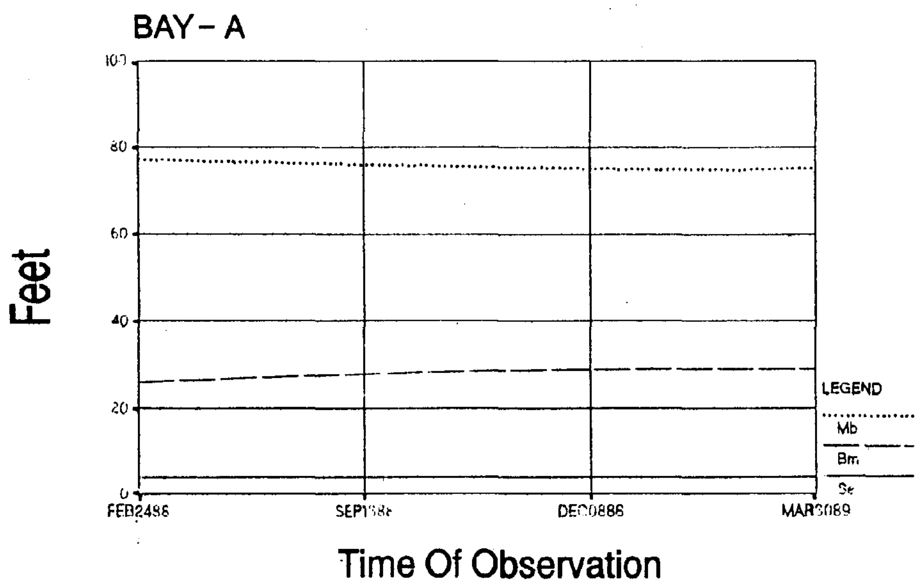


B

Figure 36. Drummonds Field - Representative Profiles.



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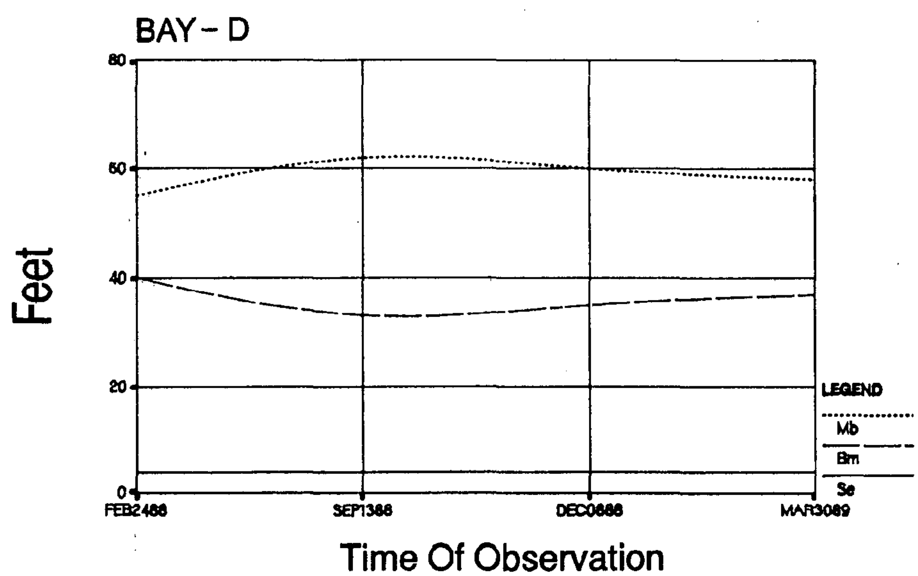


B

Figure 37. Drummonds Field - Representative Parameters.

Table 8. Parameters for Drummonds Field*
March 1989

Breakwater/Bay	L_B	G_B	X_B	h_B	F_B	M_b	T_e	S_e	B_I	B_M	T_A	T_W
Downdrift								2.2	10	13		
Breakwater 1	90		100	2.0	2.3		0.2	4.5	5	100		31
Bay A		180				75		4.0	5	29		
Breakwater 2	90		95	2.7	2.0		0.0	3.7	10	103		17
Bay B		180				65		4.3	5	43		
Breakwater 3	90		105	2.4	2.0		-0.2	4.5	5	99	3	
Bay C		220				97		4.0	10	28		
Breakwater 4	80		45	2.2	2.0		0.0	3.5	0	40		1
Bay D		200				58		4.0	10	37		
Breakwater 5	100		50	2.2	1.7		0.5	2.8	0	53		25
Bay E		60				43		3.2	5	22		
Breakwater 6	120		55	2.1	2.2		0.9	6.4	10	84		55

* All dimensions in feet.

L_B - Breakwater crest length

G_B - Breakwater gap

X_B - Distance offshore CL breakwater to original MHW

h_B - Height of breakwater from bottom at CL to MHW

F_B - Breakwater freeboard, MHW to crest

M_b - Maximum bay indentation, CL breakwater to MHW

T_e - Tombolo elevation in lee of breakwater + MHW

S_e - Backshore elevation at base of bank

B_I - Initial beach width, base of bank to MHW

B_m - Present beach width, base of bank to MHW

T_A - For unattached tombolo, MHW to CL of breakwater

T_W - For attached tombolo, tombolo width at MHW

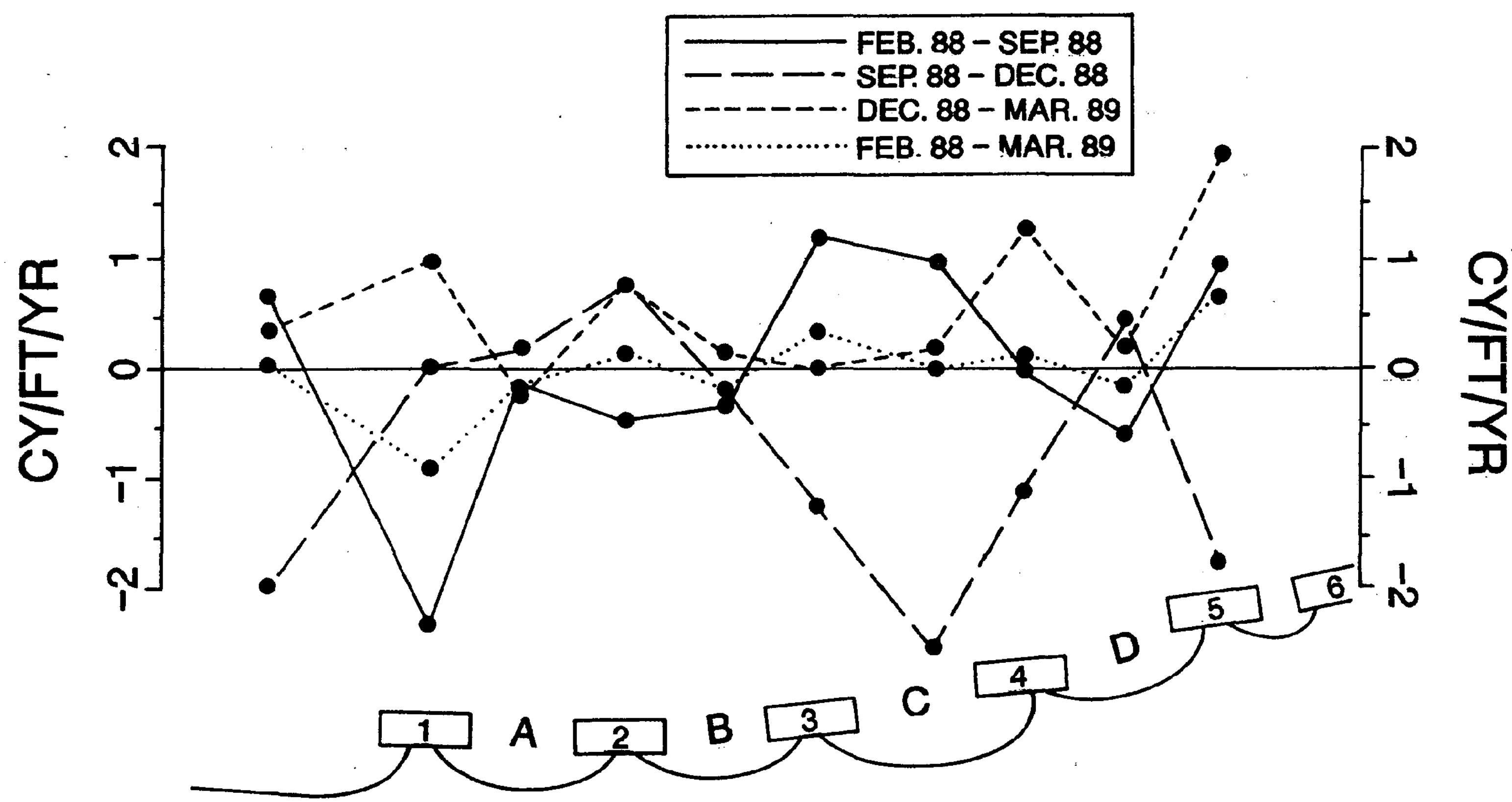
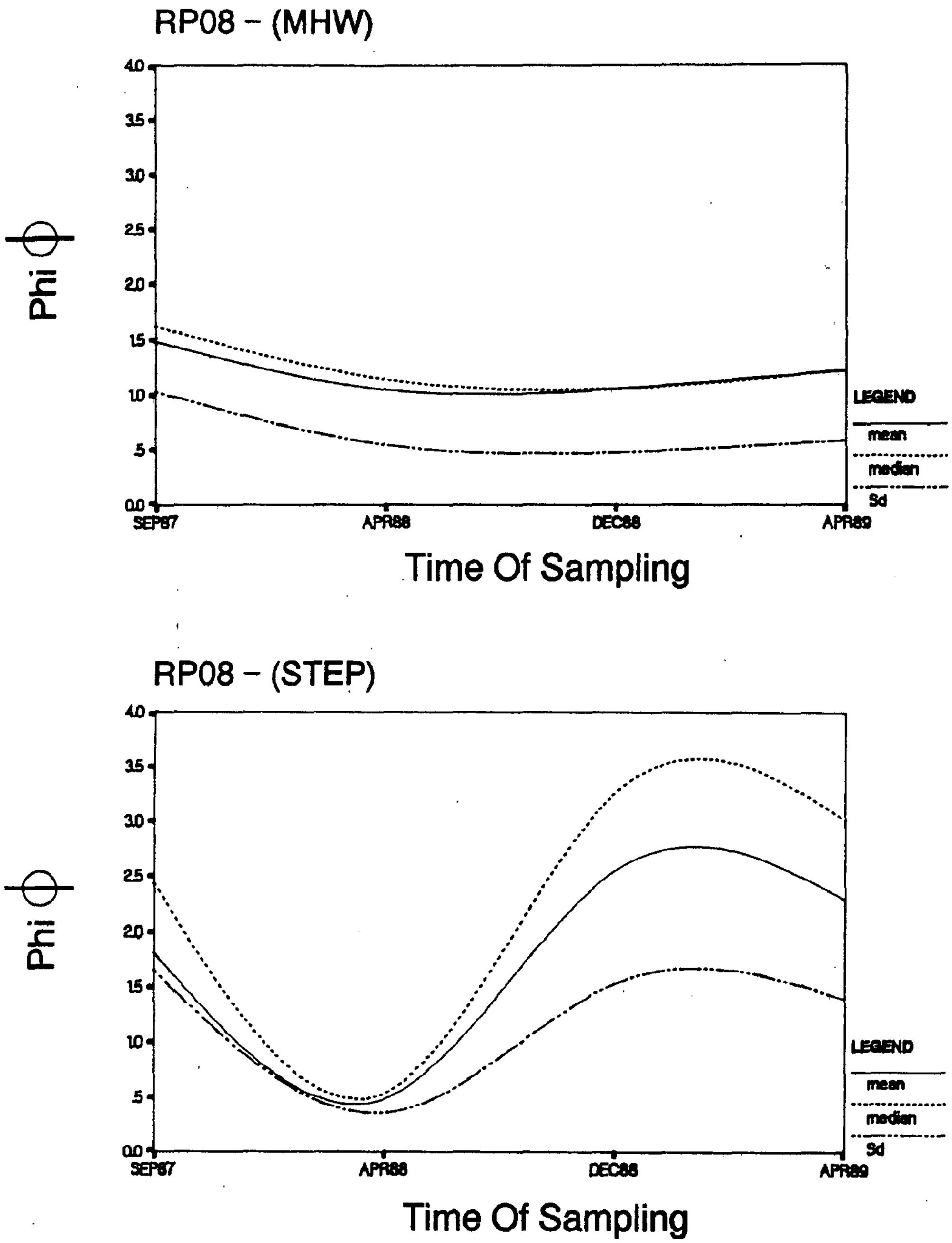


Figure 38. Drummonds Field - Beach Volume Change.



A

B

Figure 39. Drummonds Field - Representative Beach Sediment Analysis.

Waltrip, James River, James City County

The Waltrip breakwaters essentially are an upriver extension of the Drummonds Field breakwater system (Figure 32). Waltrip has the same bank type, fetch and shore orientation as Drummonds Field. Three rock breakwaters were installed in October 1987 along with about 3,000 cubic yards of beach fill. At the same time a rock extension was built onto breakwater number 6 at Drummonds Field. The upland bank was then graded. Also, a low rock groin was built on the upriver breakwater unit to keep the beach fill out of the small adjacent wetland area.

Wave Climate

The Waltrip site is exposed to the same general wave climate as Drummonds Field (Figure 33). The shore normal wave approach is reflected in the symmetrical planforms of the embayments.

Design and Construction

The Waltrip breakwater system was designed to protect the remainder of the eroding high banks along the reach upriver of the Drummond Field project. It was evident from earlier installations in this project that higher breakwaters and a higher, broader backshore would offer greater protection of the fastland banks during storm events. The result was a breakwater system composed of three rubble mound units which have crest lengths of 50 feet, crest elevations of 3.0 feet above MHW and gaps of 75 feet (Figure 40A). These are similar in design to the breakwaters at Chippokes in terms of breakwater length and gaps. However, the Waltrip breakwaters were placed about 100 feet from the original MHW line which provided enough area to develop deep pocket beaches and a broader backshore. The backshore elevation was set at 4.5 feet above MHW. The construction roads which were built to each breakwater were left as attached tombolos.

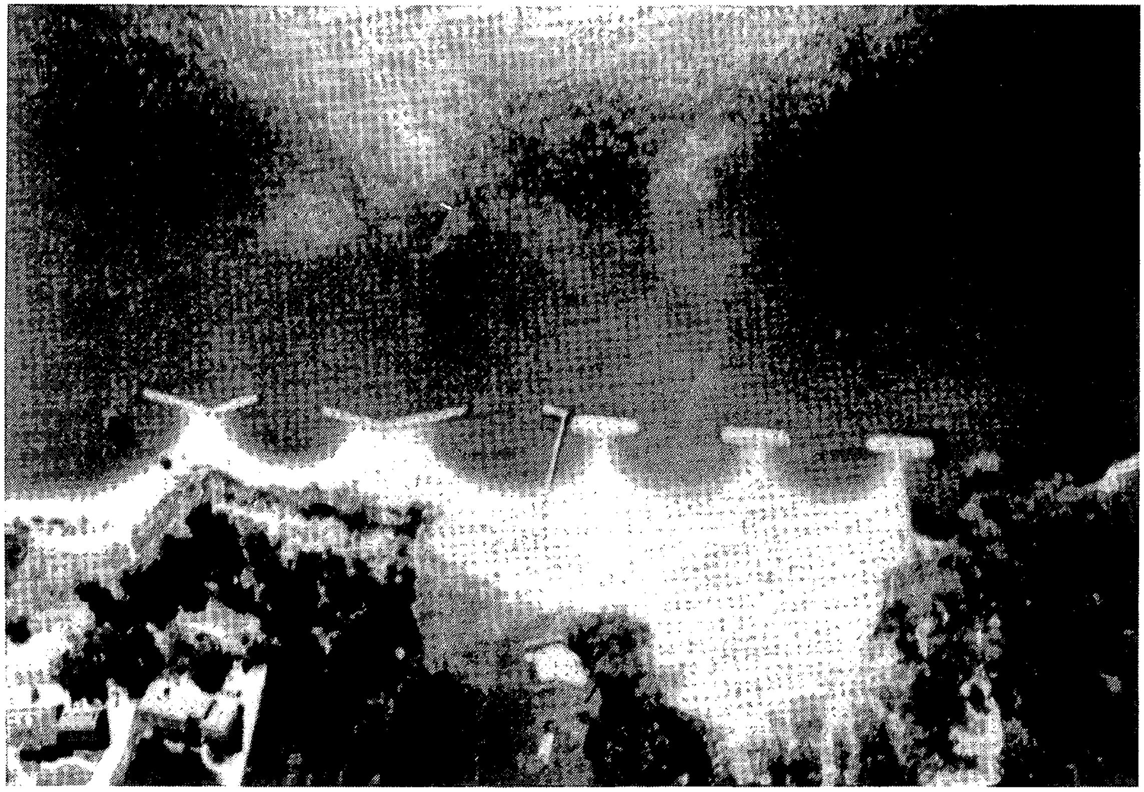
Shore Changes

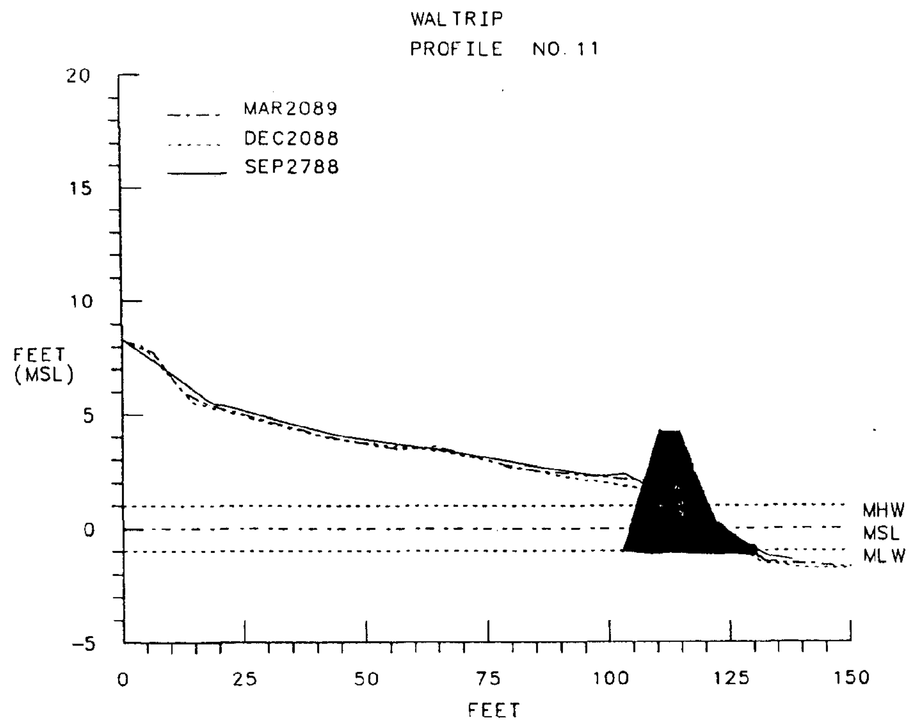
There were no pre-construction profiles established at Waltrip. The site was included in this project in August 1988 and monitoring began in September 1988. After one year, a decrease in tomoblo elevation was noted behind breakwater 2 (Figure 41A). This corresponds to a net sand volume loss in the lee of the same structure (Figure 42). A slight narrowing of the tombolos behind each breakwater unit was noted, as well as a landward shift of MHW in bay B and bay F (in the adjacent Drummonds Field breakwater system) (Figure 42).

Initial adjustments to the beach planforms occurred between the fall of 1987 and the fall of 1988. Since that time, slight changes have occurred through time to bay and breakwater parameters (Figure 43). Table 9 shows all the site parameters for the Waltrip breakwater system.

Figure 40A. Waltrip - aerial vertical. September 15, 1988. Note Drummonds Field breakwater just downriver.

Figure 40B. Waltrip - ground view, looking northwest. September 27, 1988.





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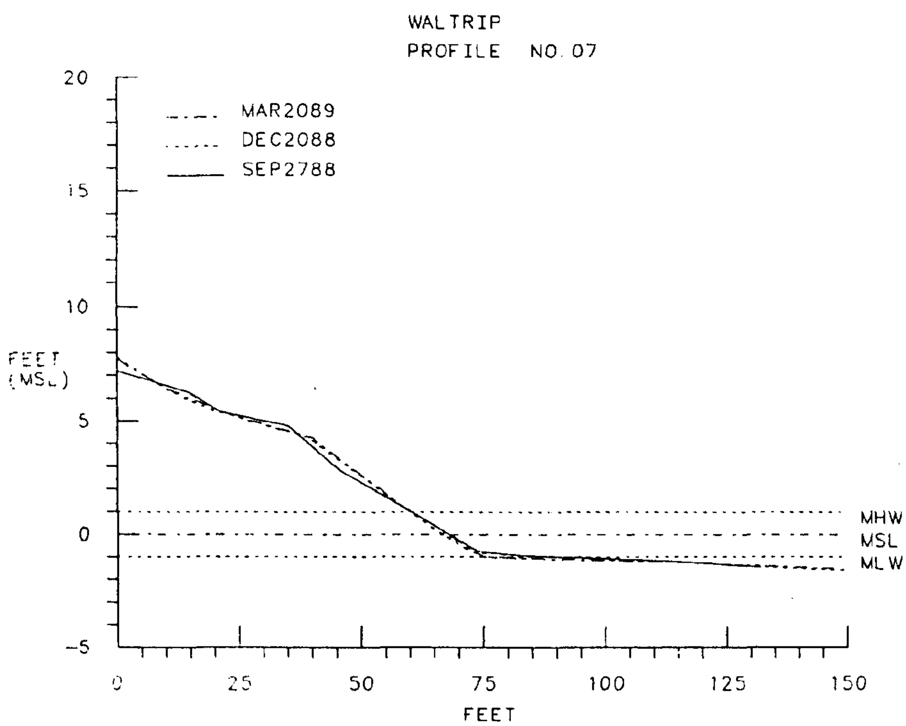


B

Figure 41. Waltrip - Representative Profiles.

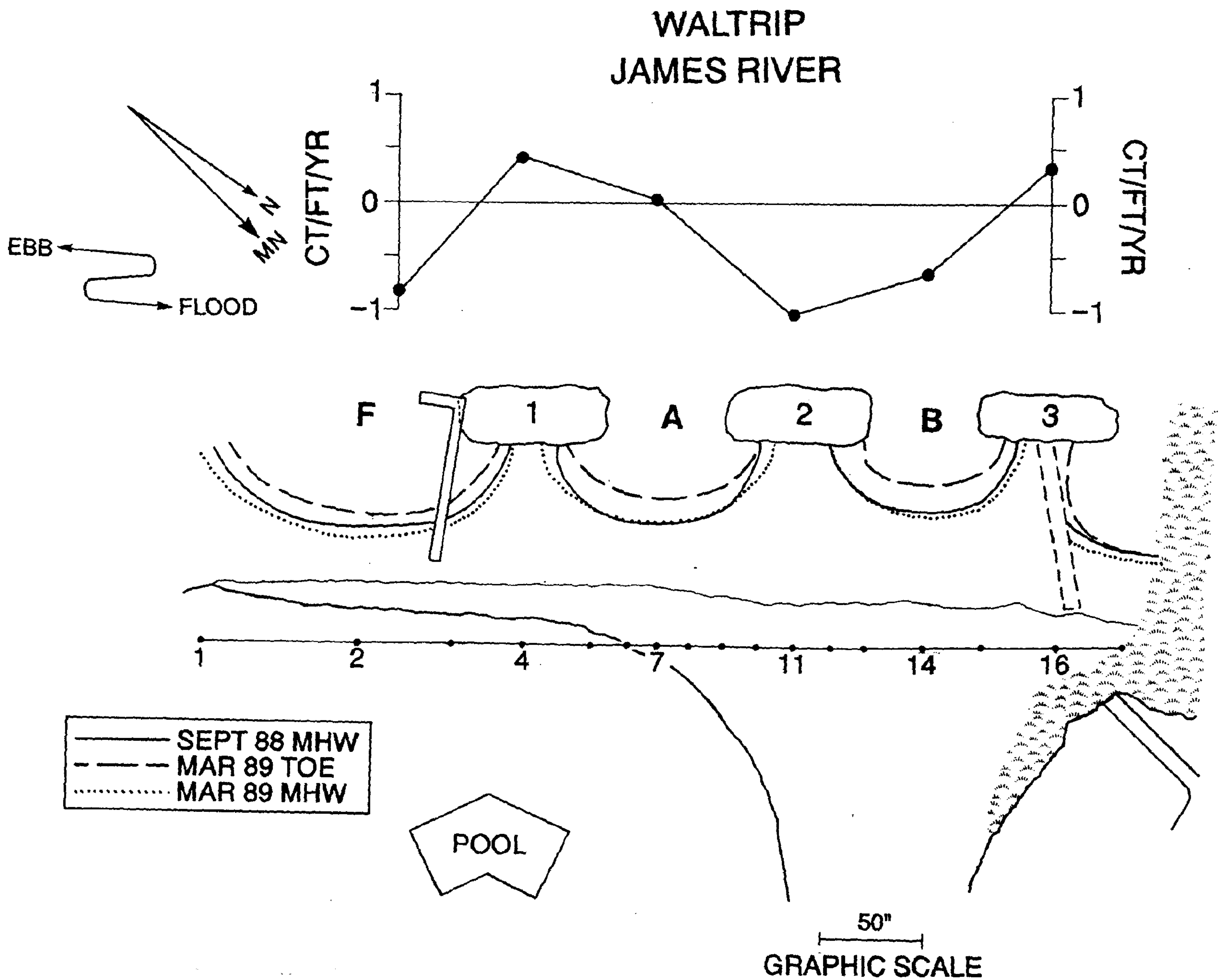


Figure 42. Waltrip - Volume Changes (top) and Base Map (bottom).

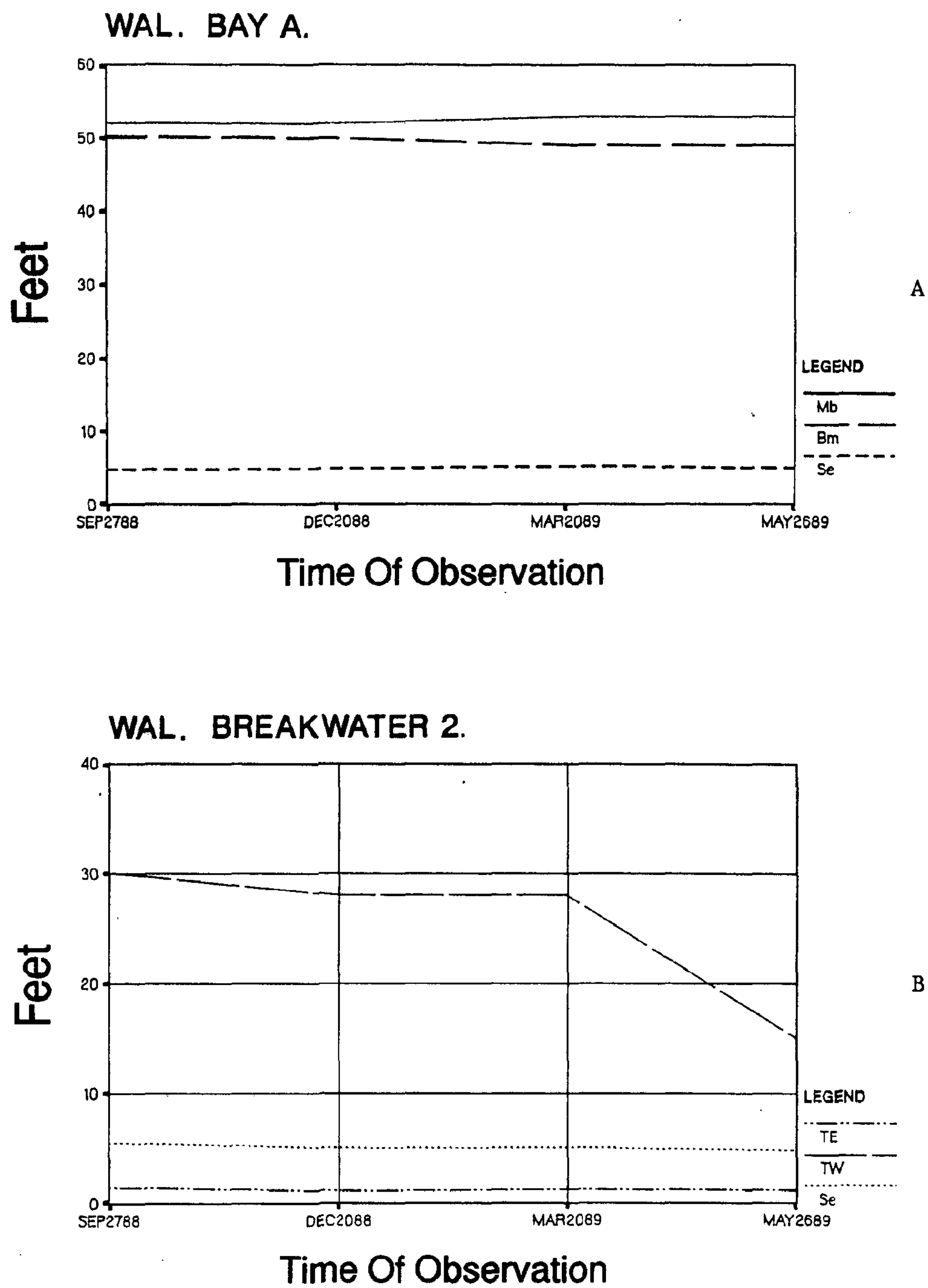


Figure 43. Waltrip - Representative Parameters.

Table 9. Parameters for Waltrip*
March 1989

Breakwater/Bay	L_B	G_B	X_B	h_B	F_B	M_b	T_e	S_e	B_I	B_M	T_A	T_W
Updrift								4.6	0	31	N/A	
Breakwater 1	50		84	2.4	3.1		1.3	4.8	0	88	N/A	13
Bay A		75				53		5.3	0	49	N/A	
Breakwater 2	50		92	2.4	3.6		1.2	5.0	0	89	N/A	28
Bay B		70				46		5.4	0	54	N/A	
Breakwater 3	50		95	2.3	3.7		0.9	2.8	0	107	N/A	15

* All dimensions in feet.

L_B - Breakwater crest length

G_B - Breakwater gap

X_B - Distance offshore CL breakwater to original MHW

h_B - Height of breakwater from bottom at CL to MHW

F_B - Breakwater freeboard, MHW to crest

M_b - Maximum bay indentation, CL breakwater to MHW

T_e - Tombolo elevation in lee of breakwater + MHW

S_e - Backshore elevation at base of bank

B_I - Initial beach width, base of bank to MHW

B_m - Present beach width, base of bank to MHW

T_A - For unattached tombolo, MHW to CL of breakwater

T_W - For attached tombolo, tombolo width at MHW

Hog Island Headlands, James River, Surry County

Three rock headlands (breakwaters) were built on the northeast shore of Hog Island in October 1987. The Hog Island headlands site is situated within a long, shallow embayment between Hog Point and Walnut Point (Figure 24). The shoreline along the embayment is curvilinear and generally faces northeast. The historical erosion rate is 2.5 feet per year (Byrne and Anderson, 1978). The bank along the northern section is approximately 10 feet high and is composed of dredged material. As one proceeds southward, the bank's elevation decreases and becomes a low (3 ft) clayey fastland. At the southern end of the site there is a marsh fringe which acts as a low, erosion-resistant headland.

In the early 1960s, large concrete blocks were placed along 150 feet of the shoreline at MLW on the north end of the site. There was a small, erosion-resistant bank midway between the blocks and the marsh headland. These features segmented the shore into two embayments (Figure 44A). Historical aerial photography shows very slight changes in shore orientation and there are no significant offsets often caused by oblique angles of wave approach. Thus a general, shore normal, long term wave-climate was indicated from the shore morphology.

Before construction, the initial beach width from the MHW line to the base of the bank varied from 0 to 20 feet and the sand layer at MHW was approximately 1 foot thick. The beach is composed of medium to coarse sand overlying stiff brown clay. Another erosion resistant, clayey bank occurs just before the downriver marsh headland. This clay bank is a small headland and marks the downriver end of the second bay.

Wave Climate

The average fetch along the Hog Island headlands shore is 2.5 nautical miles. The site is exposed to winds from the north northwest to the southeast (Figure 45). The shoreline is oriented normal to the northeast, which supports the theory of a dominant shore normal wave approach.

Design and Construction

The design of Hog Island headlands was based on the geomorphic expression of the shore. The existing protuberances caused by the concrete blocks and erosion resistant banks were designated points to construct rock headlands. The system consists of three rubble-mound breakwaters (Figure 44B). Breakwater 1 has a 150-foot crest length, a 4-foot crest width and is 3.5 feet above MHW (mean tide range = 2.1 ft). Breakwaters 2 and 3 have 100-foot crest lengths, 4-foot crest widths and are 2.0 feet above MHW. Breakwater 1 was designed with larger dimensions to provide greater protection for the high bank and nearby service road. Breakwater 1 was placed at -1.0 MLW, while breakwaters 2 and 3 were placed at 0.0 MLW.

Approximately 2,400 cubic yards of fill was placed on Hog Island headlands. This came from the same pit in Smithfield where fill was obtained for the Hog Island breakwaters. The bank behind breakwater 1 was graded and approximately 1,200 cubic yards of fill was emplaced. Approximately 600 cubic yards were placed behind each of breakwaters 2 and 3. The fill was put behind each structure as a fully attached tombolo. The new beach fill was placed at an elevation of about 3.0 feet above MHW at the backshore.

Shore Changes

The most noticeable changes to the shore planform at Hog Island headlands were to the sides of the tombolos. The area in the lee of each structure was almost filled to capacity. Adjustments to the headlands and bays are seen in Figure 46. There has been a slight loss of beach fill to the system since the initial installation.

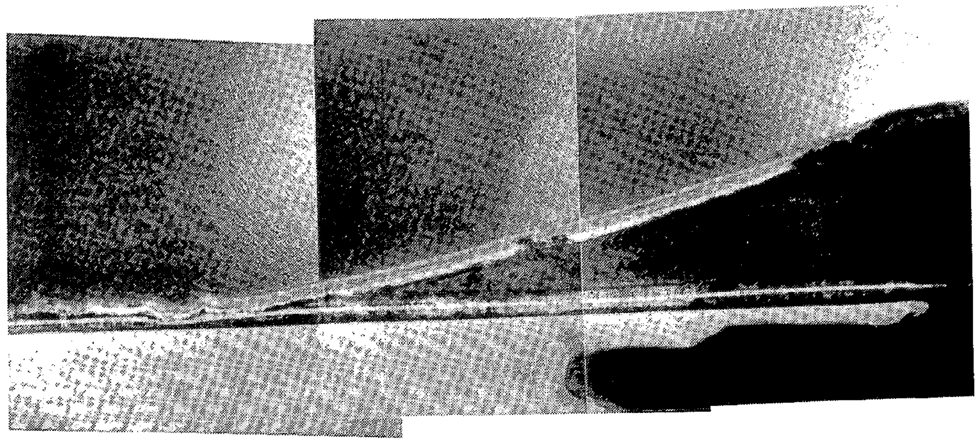
The April 1988 storm completely flooded the tombolos behind breakwaters 2 and 3, but only partially covered the river side of the tombolo behind breakwater 1. The low bank downriver of profile 10 was flooded back to the high bank by the service road. The intertidal beach was flattened along the embayed shorelines but subsequently recovered (Figure 47B). Little sand appeared to be lost offshore, but as of March 1989, sand began shifting downriver around the outside of each headland. This is probably due to the lack of strong northeast winds during the winter of 1989. The beach was more under the influence of north and northwest wind conditions (NOAA, 1989). The generally very flattened, shallow, symmetrical bays attained a slight log-spiral component against the downriver sides of each tombolo. The status of site parameters for Hog Island headlands is shown in Table 10.

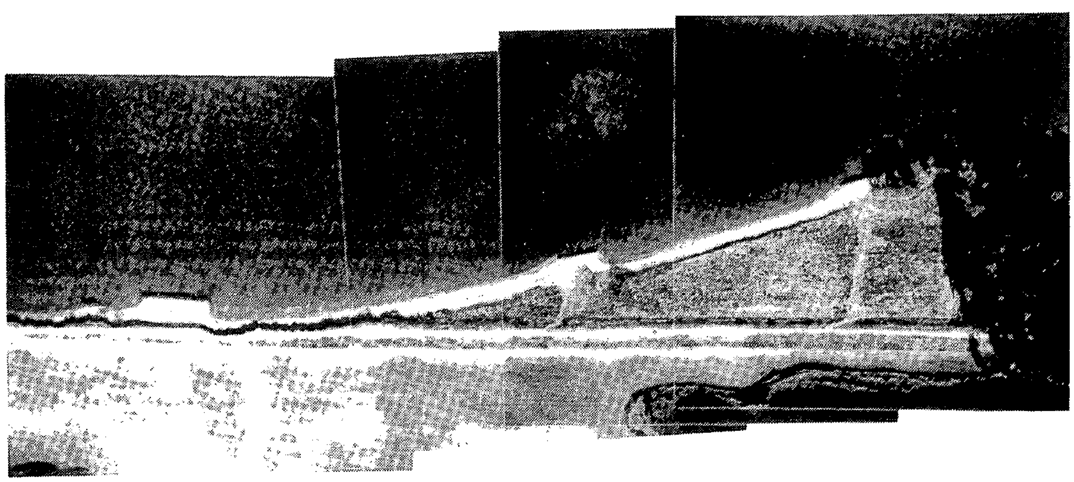
Sediments

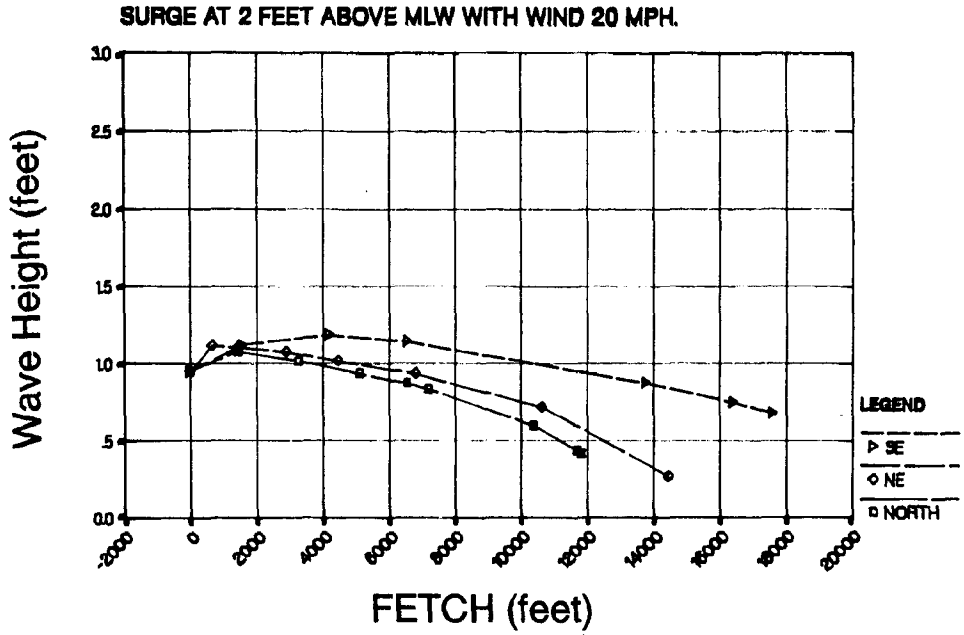
Beach sediments showed a marked change due to the April 1988 northeaster (Figure 48). Upper beach sands became coarser and less well-sorted while the lower beach or step became finer and better sorted. Subsequent trends show significant fining of the step sediments.

Figure 44A. Hog Island Headlands - aerial vertical. June 18, 1987. Pre-construction.

Figure 44B. Hog Island Headlands - aerial vertical. December 19, 1988. Post-construction.







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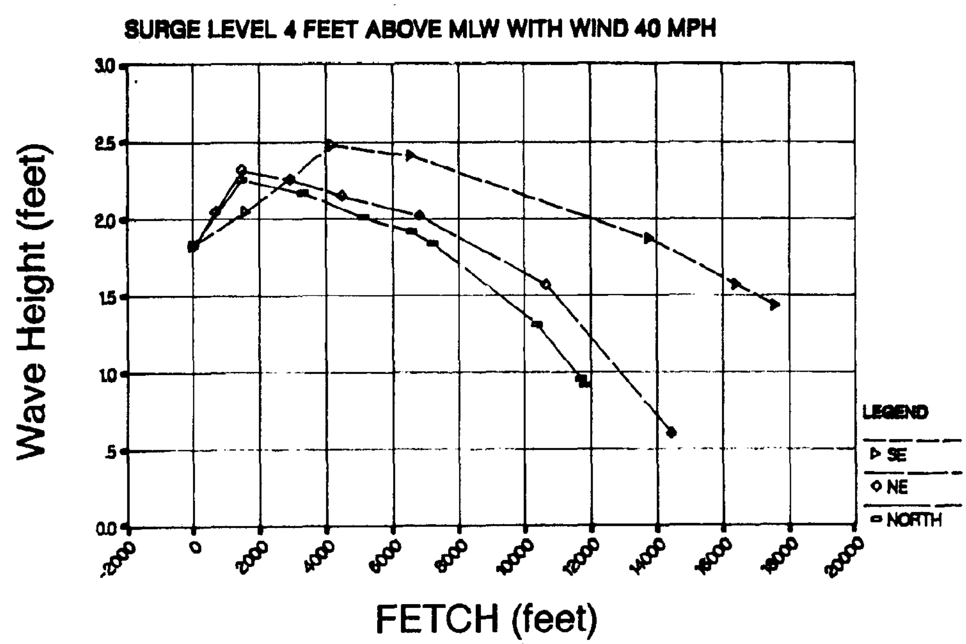


B

Figure 45. Hog Island Headlands - Wave Climate.

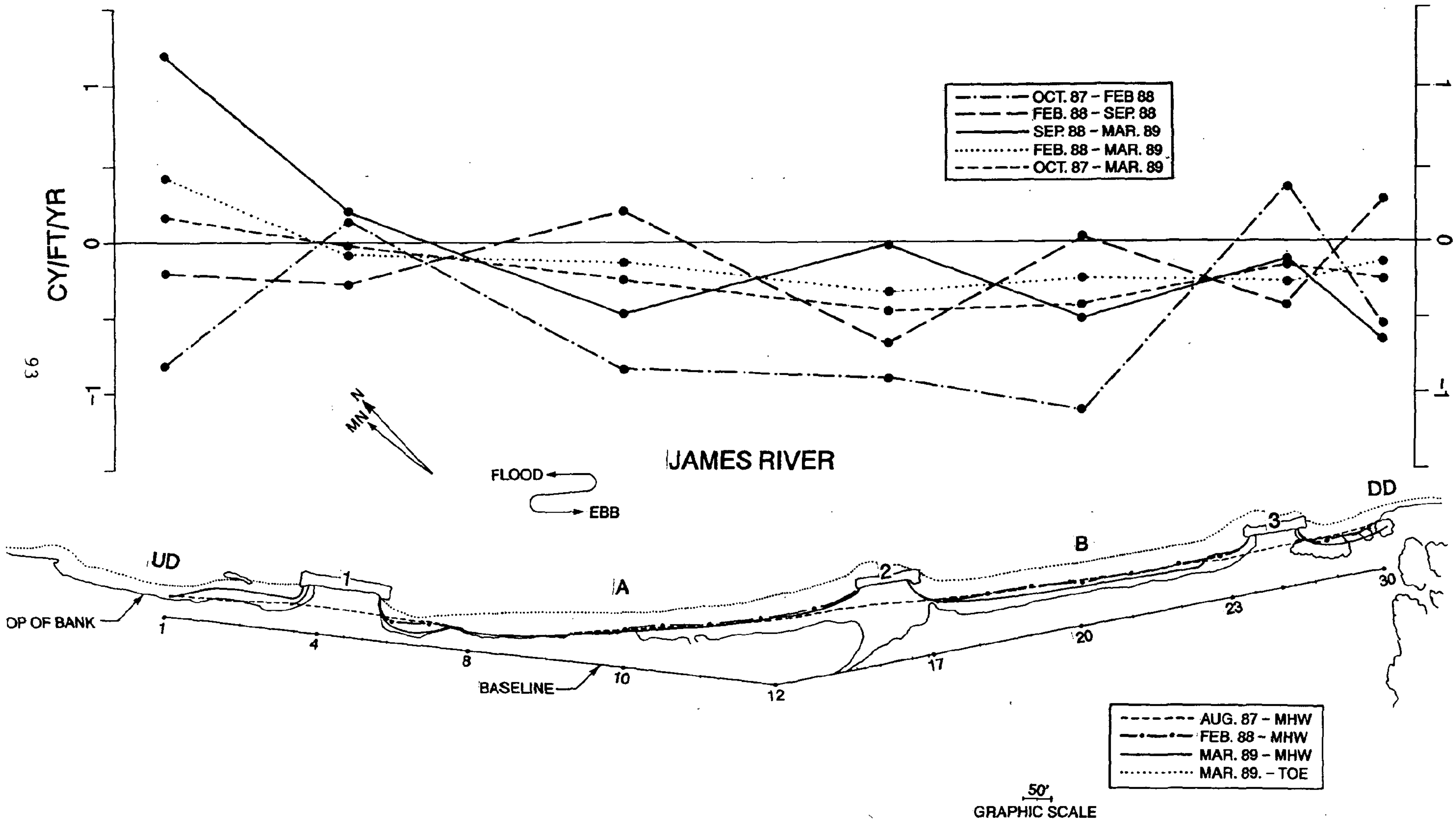
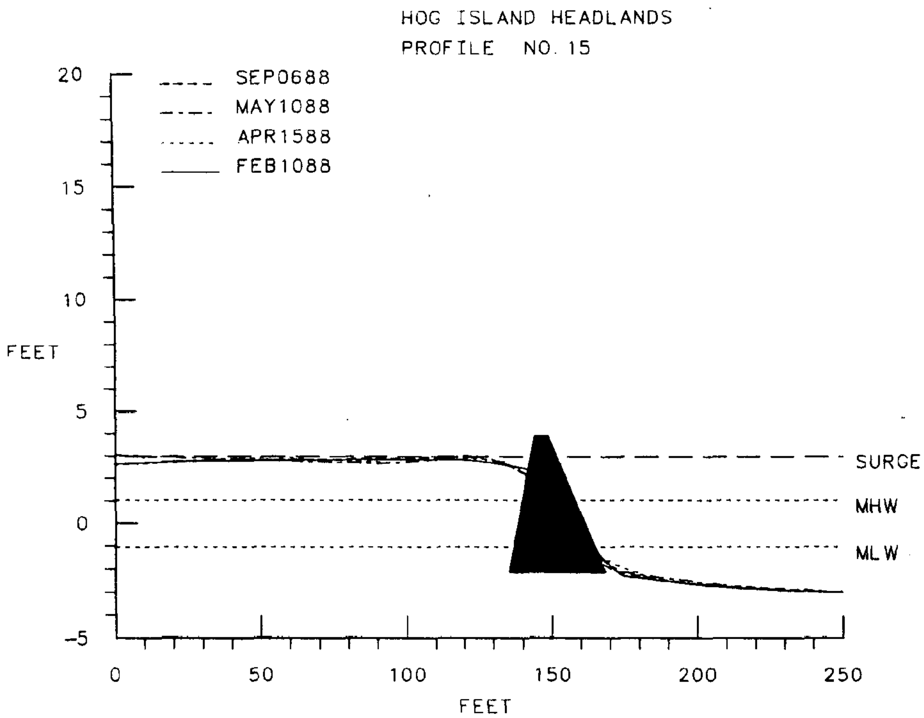


Figure 46. Hog Island Headlands - Volume Changes (*top*) and Base Map (bottom).



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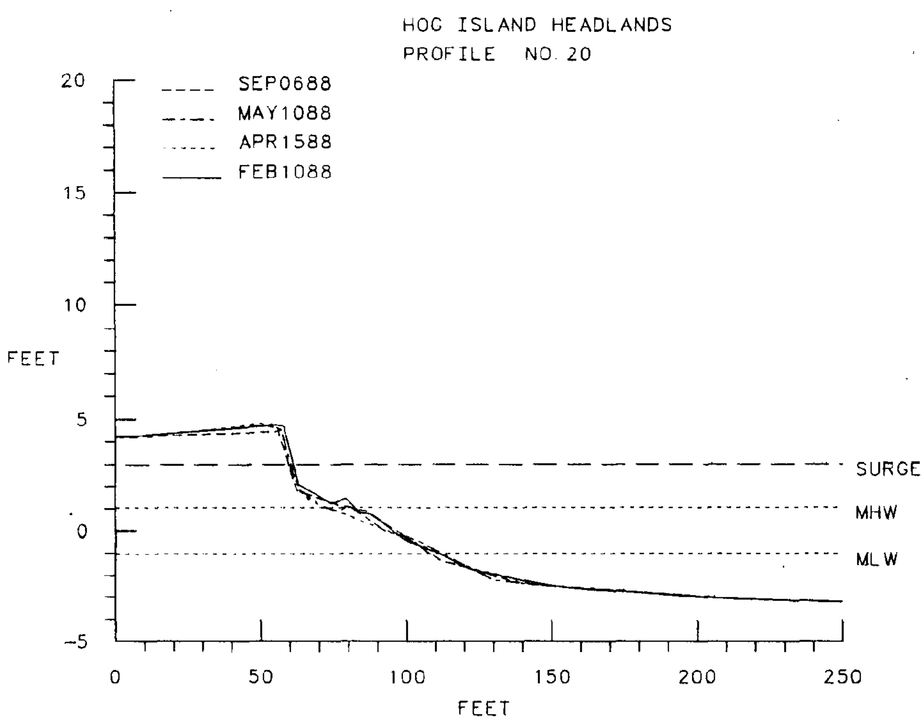


B

Figure 47. Hog Island Headlands - Representative Profiles.

Table 10. Parameters for Hog Island Headlands*
March 1989

Breakwater/Bay	L_B	G_B	X_B	h_B	F_B	M_b	T_e	S_e	B_I	B_M	T_A	T_w
Updrift						?		0.5	6	3		
Breakwater 1	150		45	3.4	3.6		1.7	3.2	19	57	150	145
Bay A		910				77.5		0.0	10	0		
Breakwater 2	100		40	2.8	2.7		1.0	2.9	12	46	100	95
Bay B		600				54.5		0.8	17	12		
Breakwater 3	100		38	2.8	2.6		1.0	2.9	15	50	90	96
Downdrift						?		1.0	5	16		

* All dimensions in feet.

L_B - Breakwater crest length

G_B - Breakwater gap

X_B - Distance offshore CL breakwater to original MHW

h_B - Height of breakwater from bottom at CL to MHW

F_B - Breakwater freeboard, MHW to crest

M_b - Maximum bay indentation, CL breakwater to MHW

T_e - Tombolo elevation in lee of breakwater + MHW

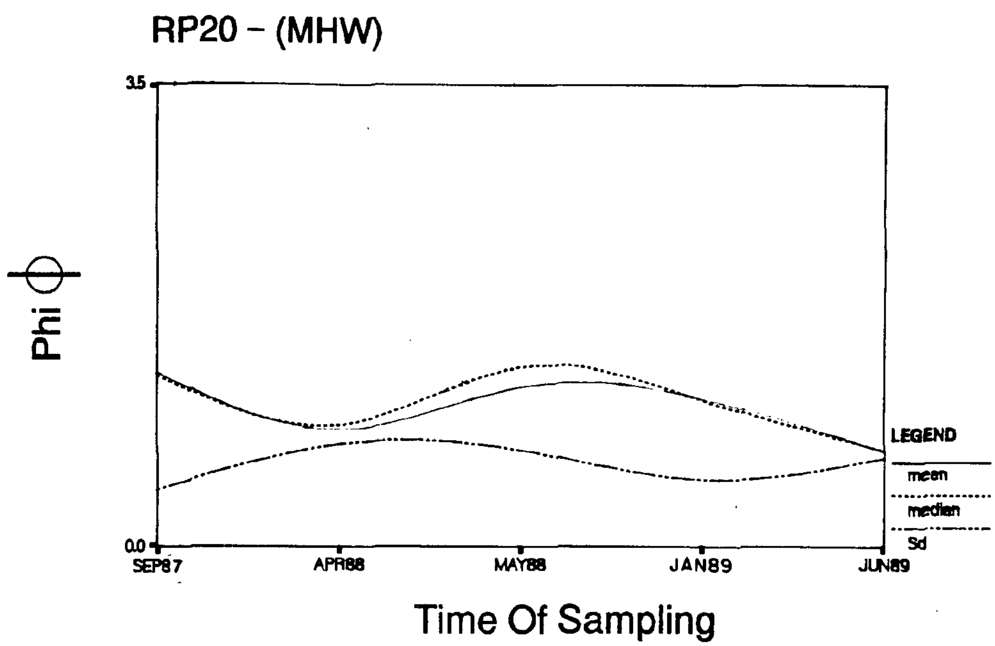
S_e - Backshore elevation at base of bank

B_I - Initial beach width, base of bank to MHW

B_m - Present beach width, base of bank to MHW

T_A - For unattached tombolo, MHW to CL of breakwater

T_W - For attached tombolo, tombolo width at MHW



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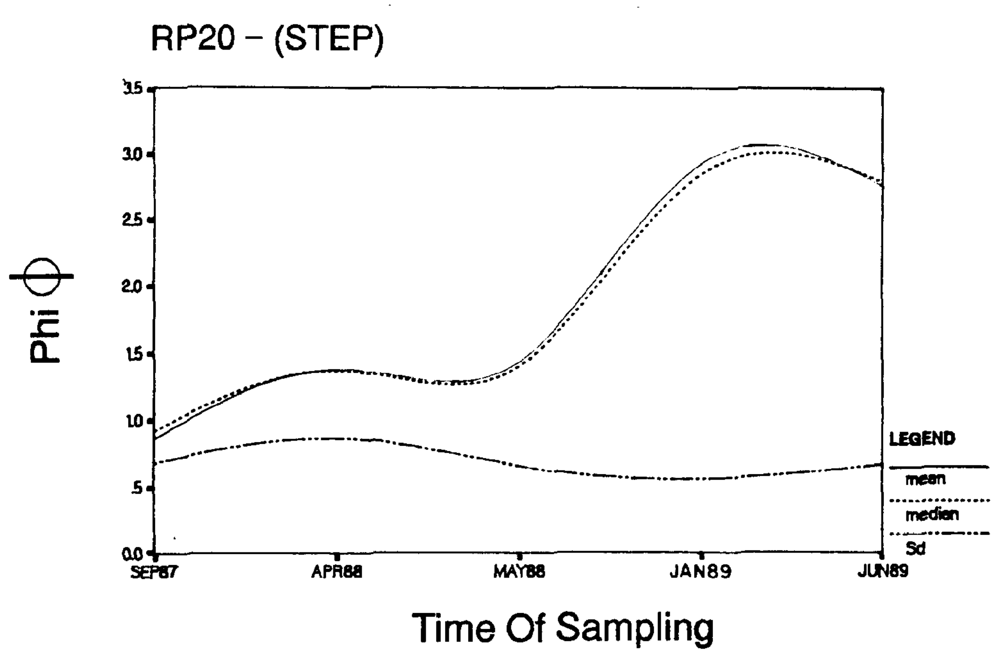


B

Figure 48. Hog Island Headlands - Representative Beach Sediment Analysis.

Yorktown Bays, York River, York County

The Yorktown Bays consist of three pocket beaches located about one mile downriver from the George P. Coleman Bridge at Yorktown, Virginia (Figure 49). The Yorktown Bays are on the property of the National Park Service's Colonial National Historical Park. This site represents estuarine beaches which have been relatively stable over a long period of time.

The Yorktown Bays have evolved over the past 50 years into three pocket beaches. The headlands separating each bay beach are composed of a highly indurated, shelly marl of the Yorktown Formation. The headlands are interfluves with banks approximately 80 feet above MSL. The bay beaches have developed in the adjacent drainages. The headlands were hardened with rock revetments in the early 1960s and reinforced in 1979. This created stable headlands and the beaches evolved into their present configuration (Figure 50A).

The Yorktown Bays are treated as three separate sites with three separate base lines. The bays are designated YB1, YB2 and YB3 (Figure 51). YB1, the largest Yorktown Bay, is approximately 400 feet long from the MHW line on each headland (Figure 50B). It is slightly crenulate shaped. The tangential section of the beach faces approximately 065°. YB2 and YB3 are smaller bays (150 ft and 190 ft, respectively) and have similar orientations. All three bays are most influenced by northeasterly winds.

Wave Climate

The Yorktown Bays face east northeast and have an average fetch across the York River of 2.2 nautical miles. However, there is a long fetch of 23 nautical miles to the east out the mouth of the York River and across Chesapeake Bay. The nearshore bathymetry moderates incoming storm waves (Figure 52). The predicted storm wave height (slightly greater than 2 feet) compares favorably to waves observed during the April 1988 northeaster.

Shore Changes

During the period August 1987 to March 1988, northwest winds were most dominant. The result was a shift of sand in each bay from the northwest to the southeast. Representative profiles of YB1 reflect this shift (Hardaway et al., 1988). This is an ephemeral situation. The general orientation of each bay aligns to the northeast over the long term as seen in historical aerial photography.

The Yorktown Bays have been observed during several northeast storms including the severe storm on November 4, 1985 and the April 1988 northeaster. YB1, with backshore elevations of 3.5 to 4.5 feet above MSL, became slightly deflated along the beach face. Beach sand was shifted back to the northwest end of the bay (Figure 53). However, no major beach cut or erosion to the backshore has been seen. The Yorktown Bays represent a unique shoreline situation in Virginia where stable, pocket beaches have evolved by a combination of geologic setting and the landowner's response to shore erosion (i.e. with the installation of riprap revetments).

Sediments

The beach on YB1 is characterized by generally well sorted medium coarse, shelly sand and gravel. The sands have been derived from historic and continued erosion of the adjacent headlands. Although riprap revetments protect the lower fourth of the headlands, the upper three quarters are exposed and actively eroding by surface runoff and pedistrian traffic.

The April 1988 storm produced the most noticeable changes in beach sand characteristics along the upper beach of YB1 in the tangential section of the bay (Figure 54A) and along the lower beach or step in the log-spiral section of the bay (Figure 54B). Here, there was a general trend toward coarser sands after the storm, followed by a return of finer sized material.

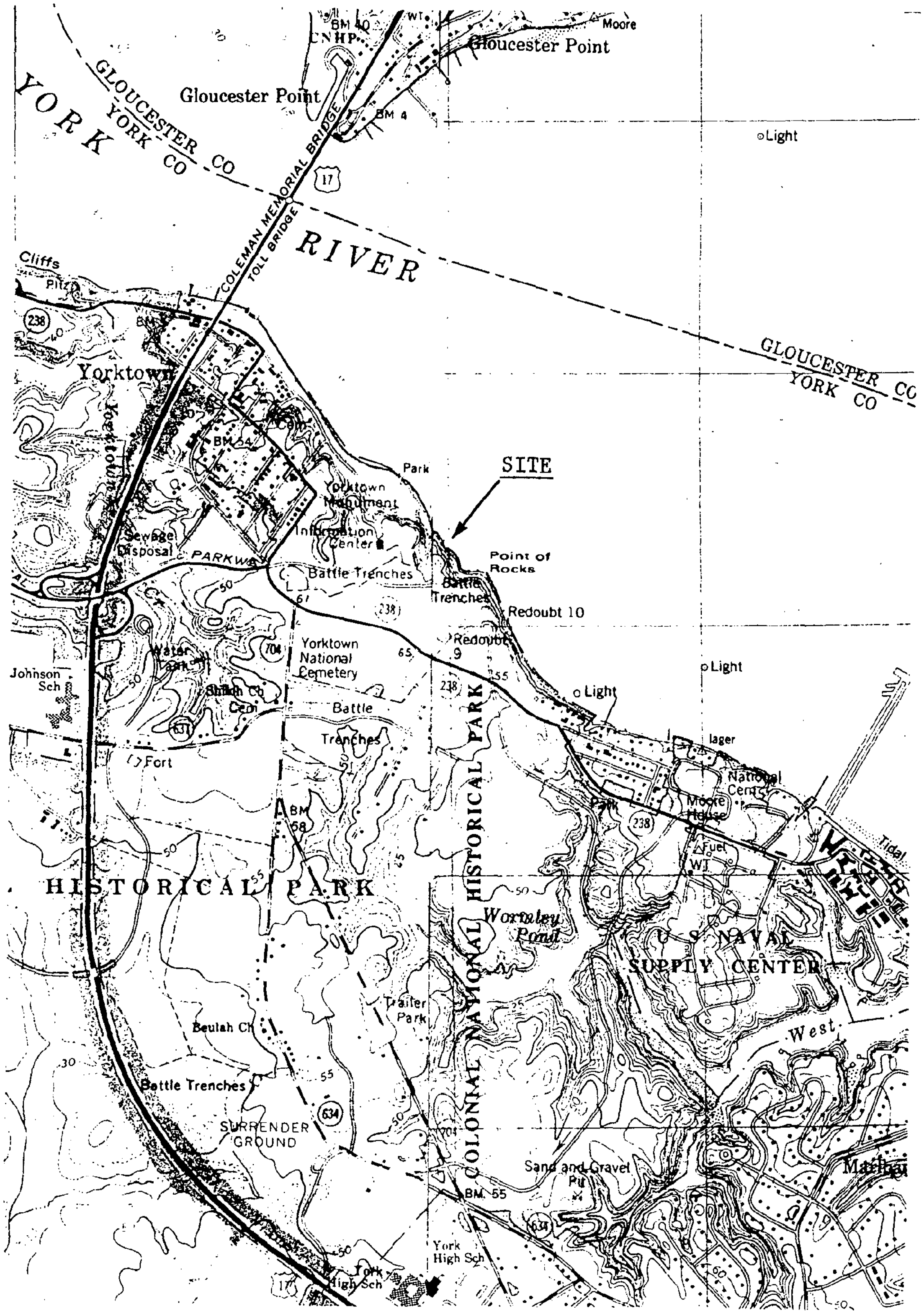
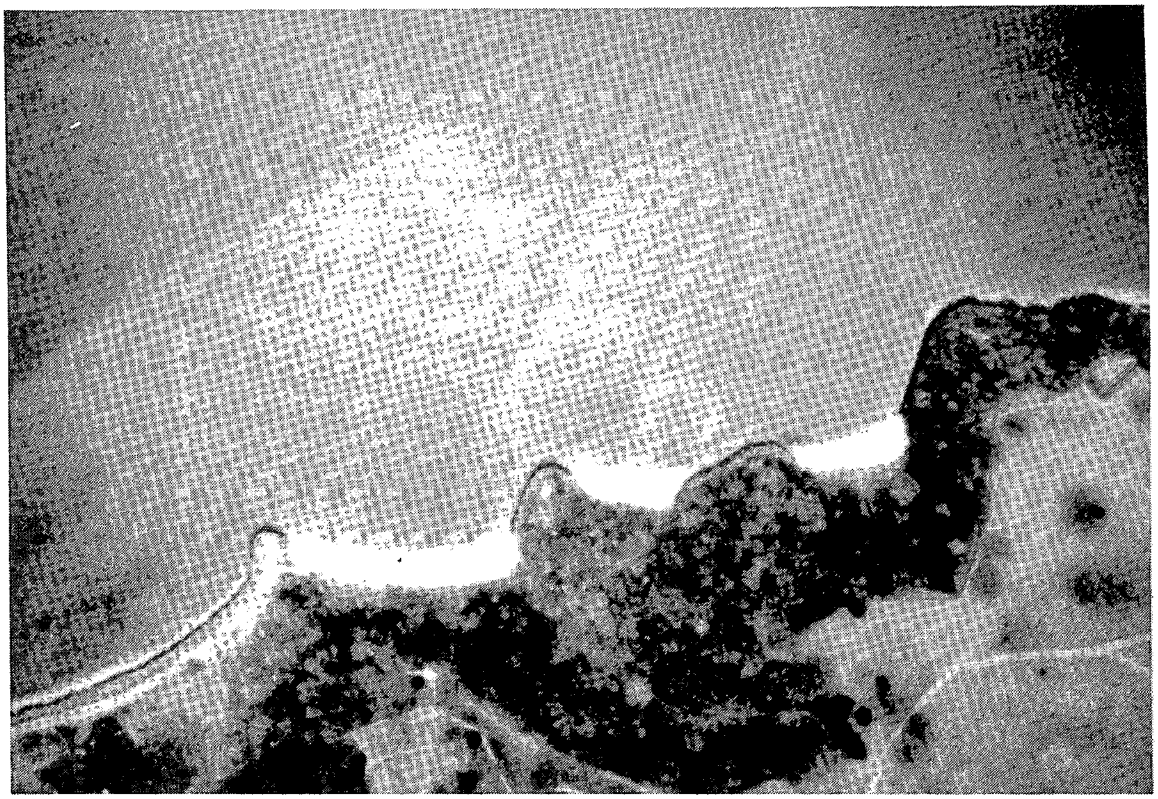


Figure 49. Yorktown Bays, York River, York County.
From Yorktown and Poquoson West 7.5 minute quadrangles.
Scale: 1 inch = 2,000 feet.

Figure 50A. Yorktown Bays - aerial vertical. March 9, 1988.

Figure 50B. Yorktown Bays - No. 1 ground view, looking east during April 13, 1988 northeaster.



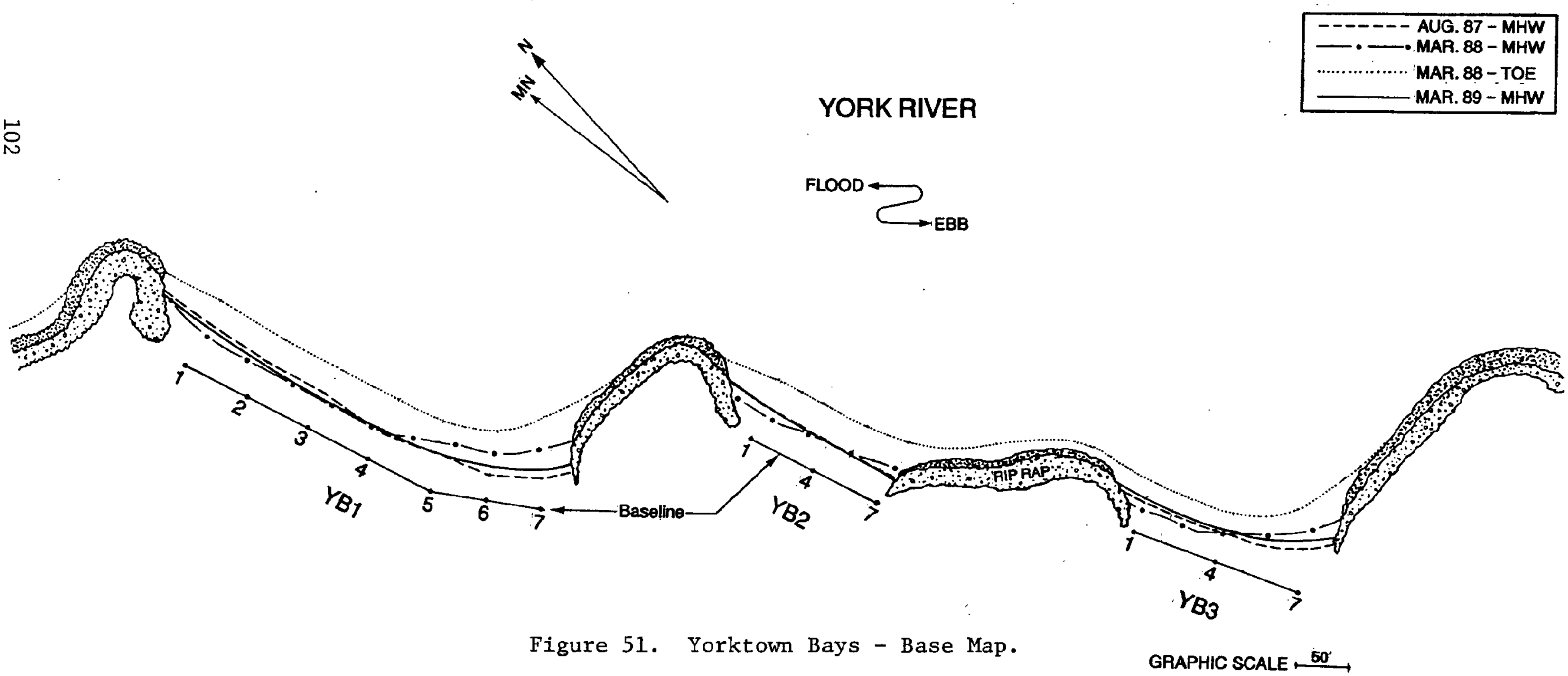


Figure 51. Yorktown Bays - Base Map.

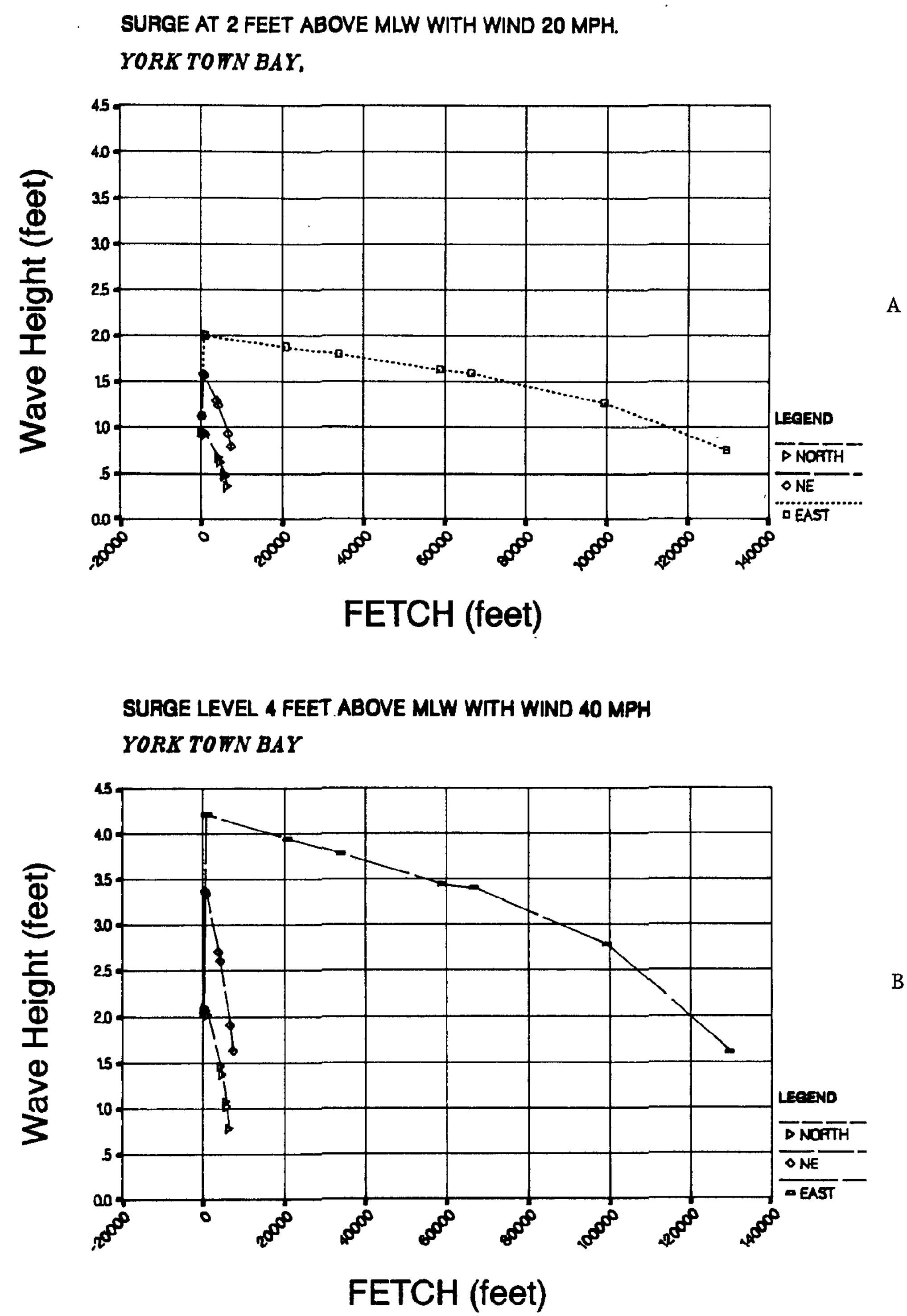
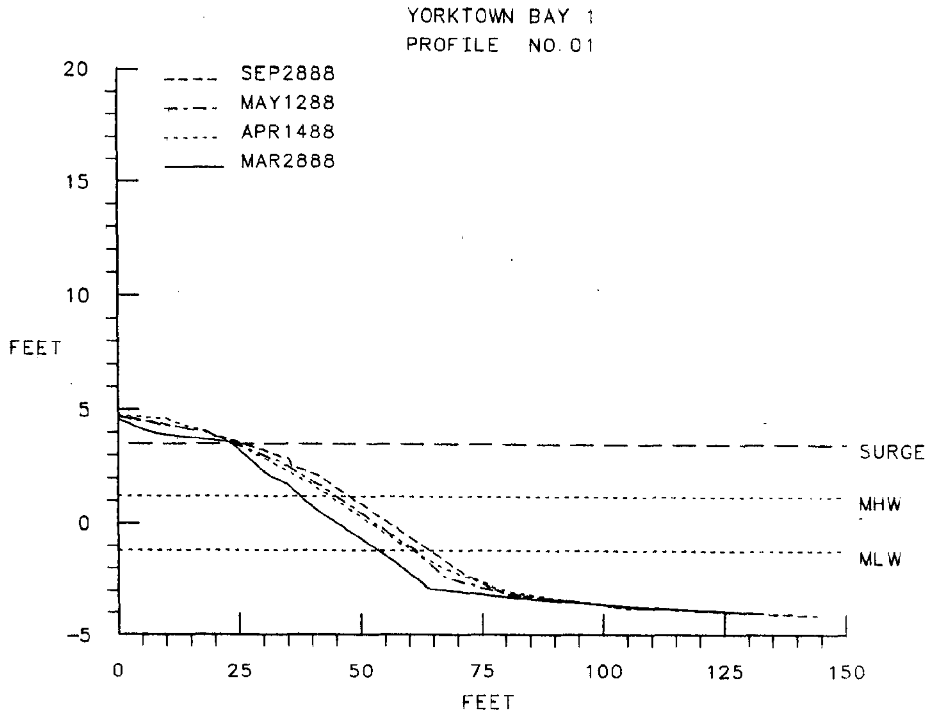


Figure 52. Yorktown Bays - Wave Climate.



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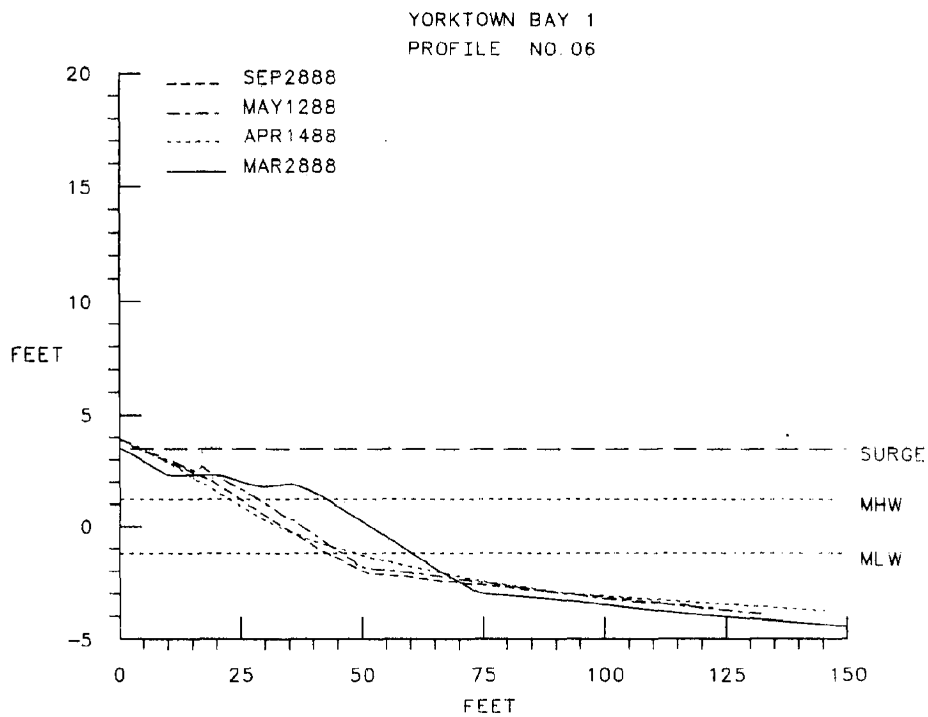


B

Figure 53. Yorktown Bays - Representative Profiles.

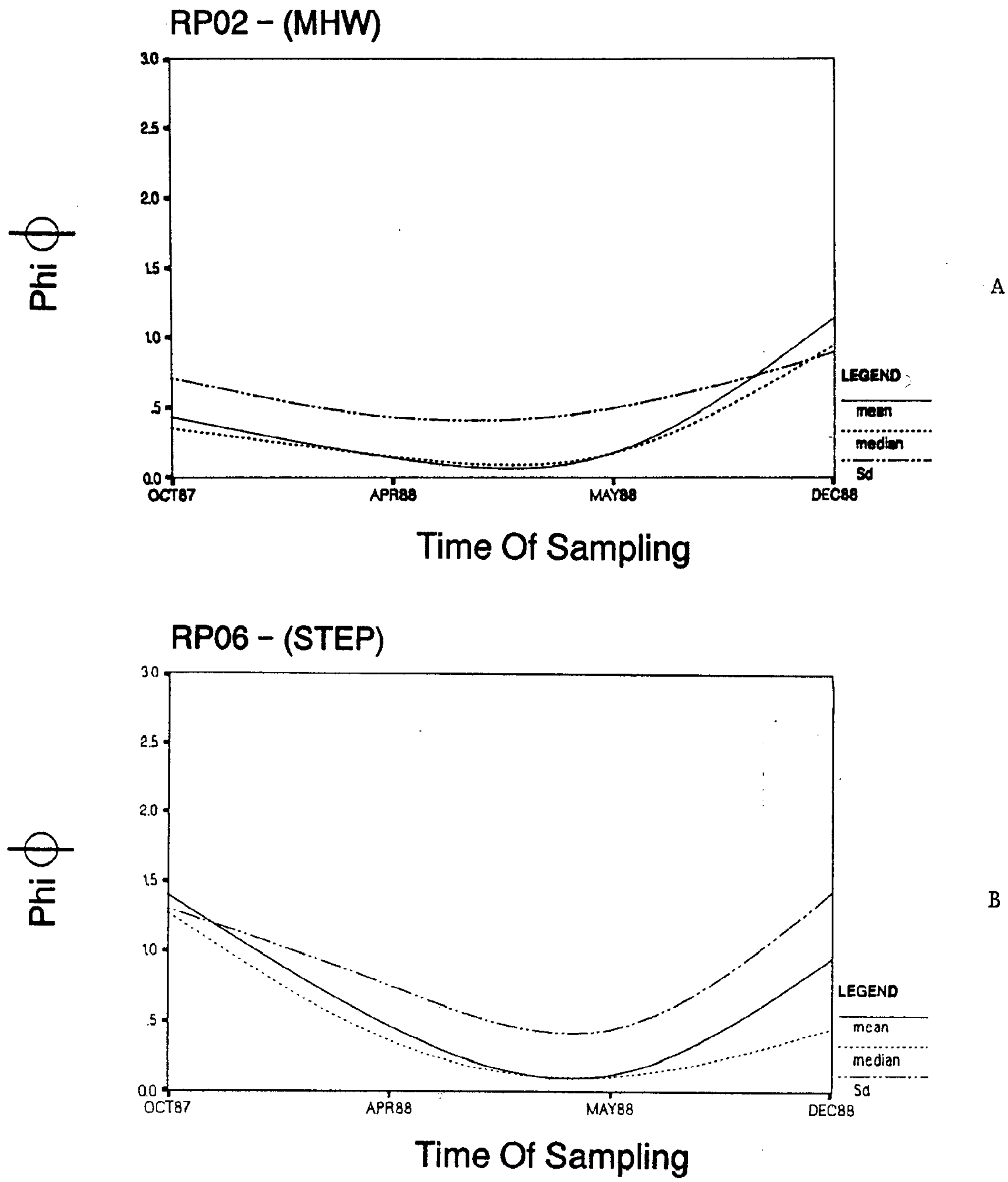


Figure 54. Yorktown Bays - Representative Beach Sediment Analysis.

Summerille, Potomac River, Northumberland County

The Summerille site is located on the Potomac River near Smith Point (Figure 55). The purpose of including the Summerille site in this project is to document the evolution of a crenulate-shaped embayment along an estuarine shore. Figure 56 depicts the positions of the 10-foot high fastland bank from 1937 to 1973. The segment in front of the Summerille house evolved into headlands after the installation of a groin field in 1967. The fastland here has an historic erosion rate of approximately 5 feet per year (Byrne and Anderson, 1978). A low sandbag sill, which was installed in front of the Staples house (downdrift neighbor) in 1975, had the effect of slowing the erosion. Consequently, a bay has evolved between the Staples house and Summerille house.

Wave Climate

The Summerille shoreline faces northeast and has an average fetch of 9.5 nautical miles. The general wave heights for different directions and surges and wind speeds are shown in Figure 57. There have been few wave observations during storm events. Consequently, the effect of the nearshore bar system on wave height and angle of approach is not known. It is felt that these bars play a significant role in the littoral processes acting upon Summerille.

Shore Changes

By 1973, the Summerille groin field had created a 40-foot offset to the southeast. In 1978, a northeast storm caused an additional 10 feet of bank loss (Anderson et al., 1983). A gabion spur, which reduced erosion immediately downdrift of the groin field, was constructed. However, the banks continued to erode further downriver. The Staples' sill was 50 feet offshore in 1978. In 1987 a short rock revetment was placed in front of the Staples house (Figure 58). This will eventually act as a small headland within the larger embayment.

The evolution of the Summerille/Staples bay sparked interest in controlling shore erosion by headland implacement and allowing the unprotected banks to evolve into what would eventually become a stable shore planform. The question becomes, "how far will the banks erode before that stable situation occurs?"

The tangential section of the bay runs from the Staples revetment northwestward along, and roughly parallel to, the surveyed baseline to profile 4, where the log-spiral curves toward the Summerille spur (Figure 59). Its orientation is generally to the north northeast which indicates seasonal wave climate at this point from a northern direction and net littoral transport southward toward Smith Point. Bank losses have been greatest in the center of the embayment since August 1986 (Figure 59). The April 1988 northeaster caused further erosion of the fastland bank, especially at profile 4 (Figure 60A). No erosion of the fastland occurred in the lee of the spur.

The beach along the embayment averages about 30 feet wide from the base of the bank to MHW. It is characterized by well-sorted, medium sands. This material is derived from erosion of the bay banks and littoral transport which brings sand into the embayment from eroding shorelines to the north.

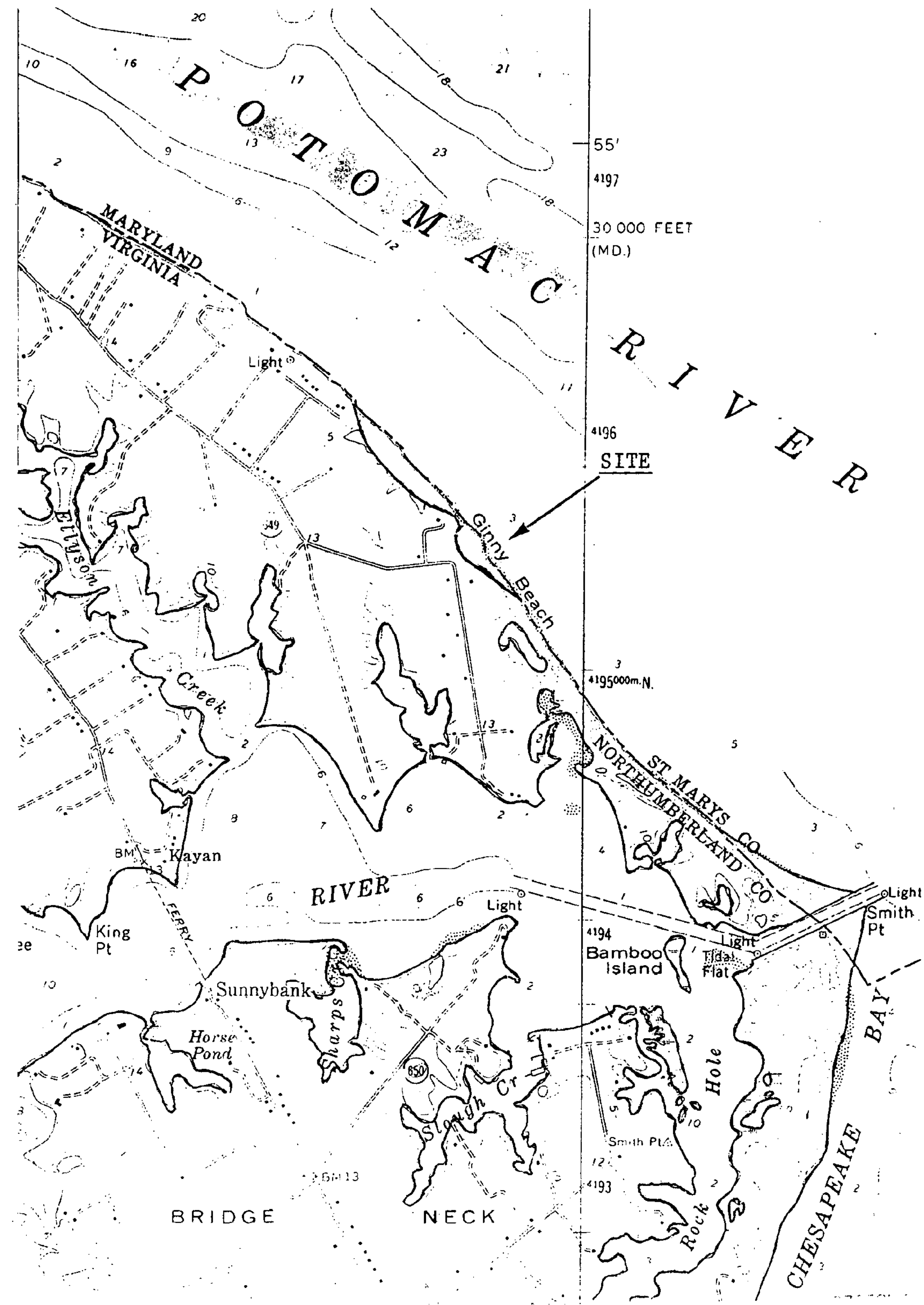


Figure 55. Summerille, Potomac River, Northumberland County.
From Burgess 7.5 minute quadrangle.
Scale: 1 inch = 2,000 feet.

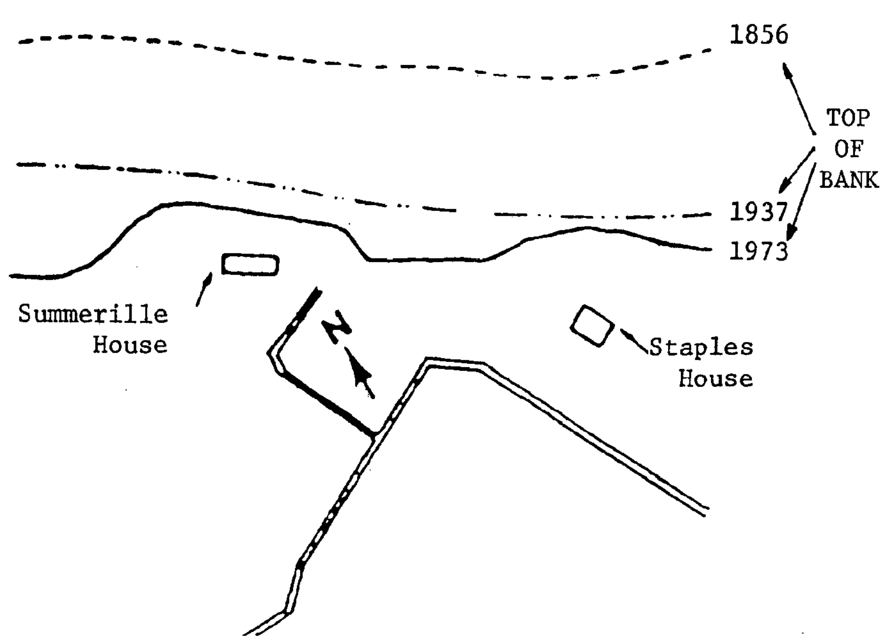
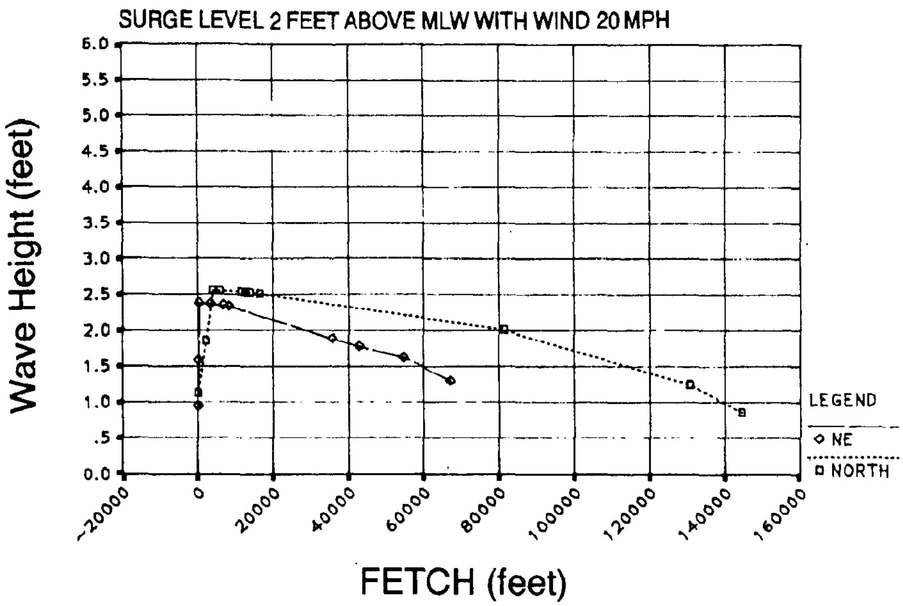


Figure 56. Summerille spur site, historical shoreline changes (after Anderson et al., 1983).



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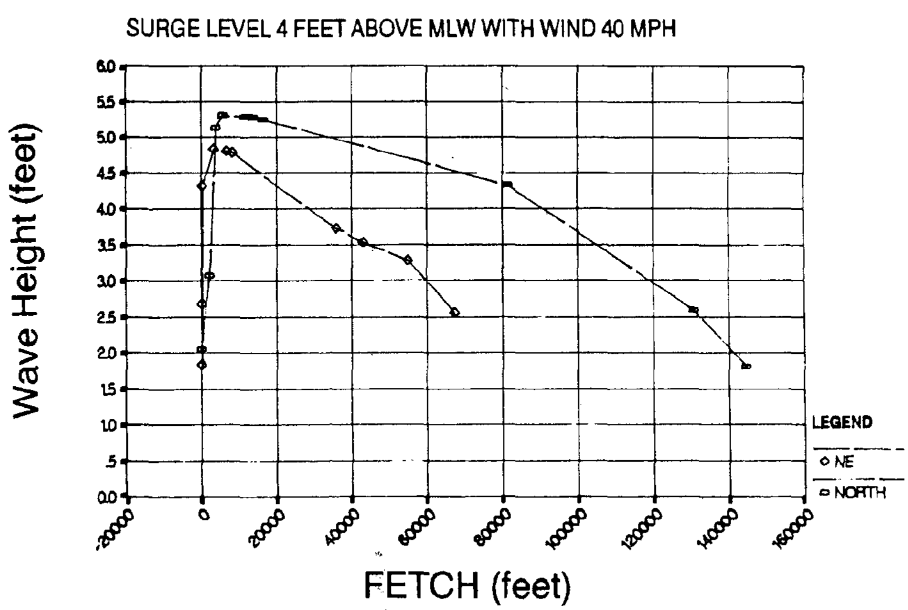
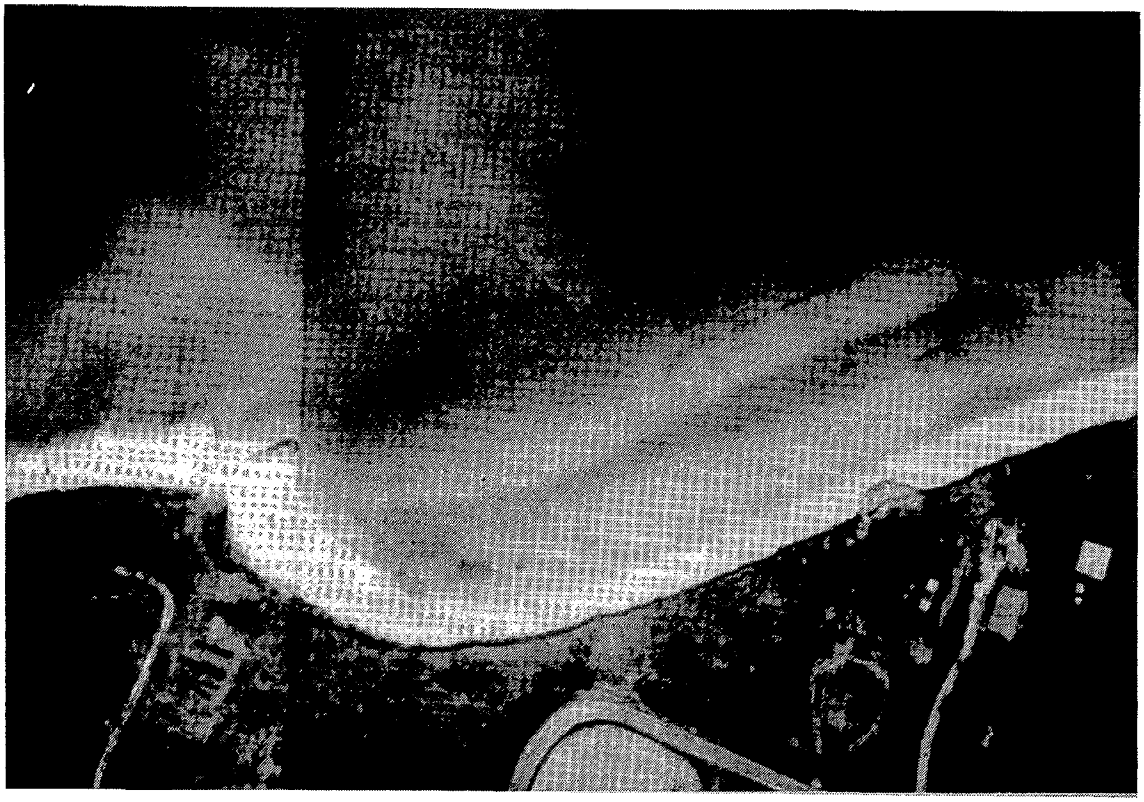


B

Figure 57. Summerille - Wave Climate.

Figure 58A. Summerille - aerial vertical. March 29, 1988.

Figure 58B. Summerille - ground view, looking west from Staples' revetment. May 13, 1988.



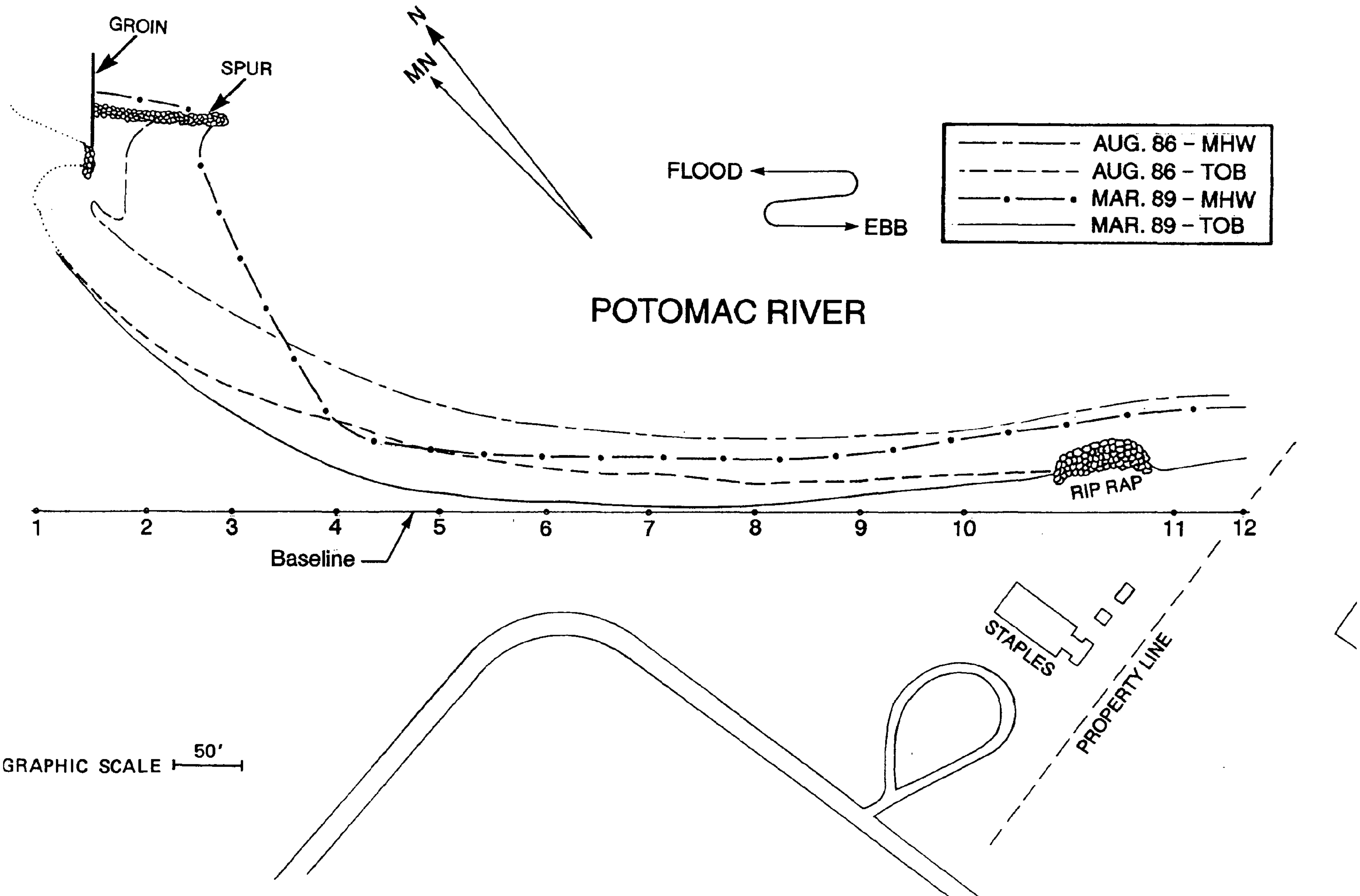
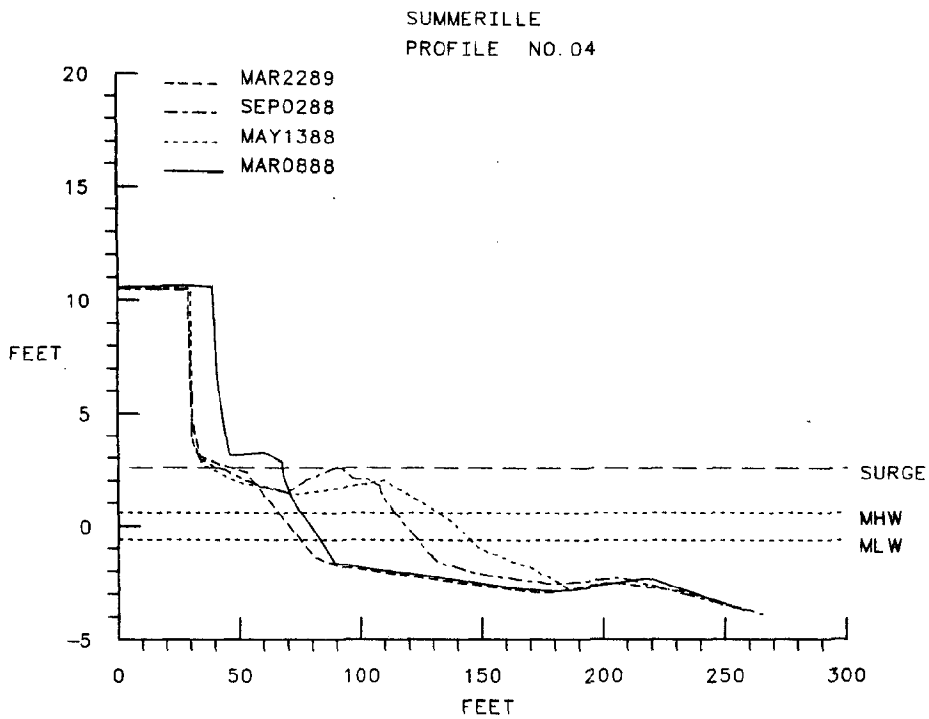


Figure 59. Summerille - Base Map.



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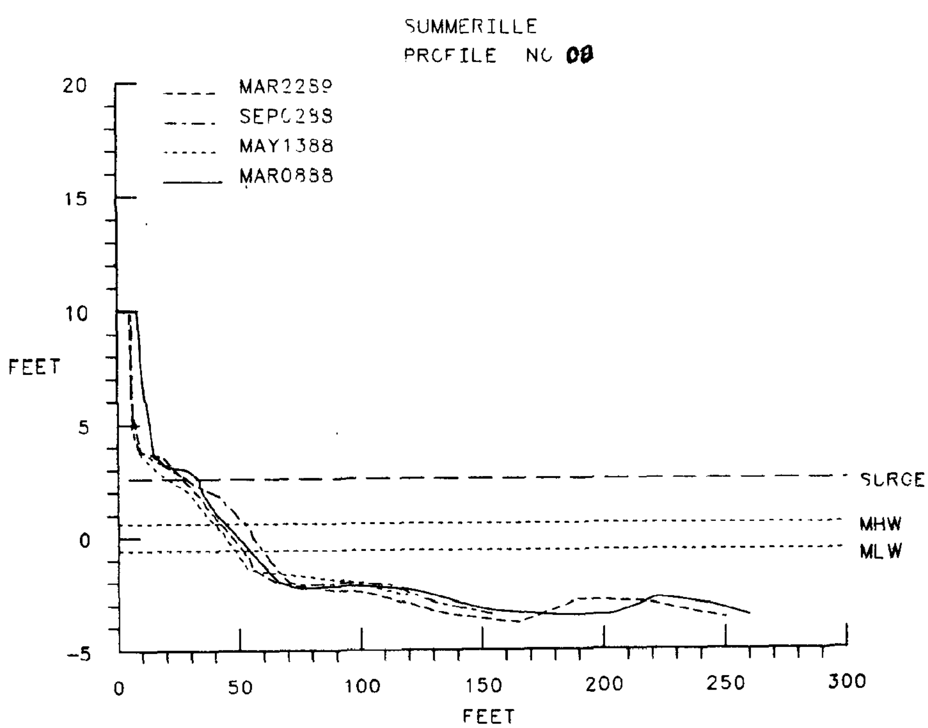


B

Figure 60. Summerille - Representative Profiles.

Results

Breakwater Sites

Results from last year's analysis of the study sites showed several general relationships of the bay/breakwater parameters. Longer, higher breakwaters have the capability to trap and maintain larger volumes of sand and thus maintain a higher, more protective backshore elevation (S_e). The relationship of the breakwater's freeboard or height above MHW (F_B) and backshore elevation can be seen in Figure 61. Generally, the greater freeboard maintains a higher backshore elevation. This is partially related to how high the salient or full tombolo is at the attachment behind the breakwater (T_e) as seen in Figure 62. Breakwater heights 2 feet above MHW (i.e. Drummonds Field, Waltrip and Chippokes) would be more conducive to a positive T_e over the long term for sites with average fetch exposures from 1 to 5 nautical miles.

The long term performance of each breakwater site can be assessed by how stable beach planforms attenuate wave action and reduce erosion along the base of the upland bank. At this point, the relative stability of the base of the bank can be evaluated in terms of bay beach width (Bm) and backshore elevation (S_e) (Figure 63). The average offshore distance (XB) of the breakwaters for each site is also plotted. Although we classify the Yorktown Bays as headlands, it seemed appropriate to plot those beach parameters along with the breakwaters due to the longevity of these stable pocket beach planforms under relatively high energy wave climate. It can be seen that the base of the bank becomes stable where Bm and S_e are greater than 30 feet and 3.0 feet above MHW, respectively. In the case of the Yorktown Bays, Bm is on the order of 40 to 50 feet. Beach fill at Drummonds Field and Waltrip provided the protective beach planform.

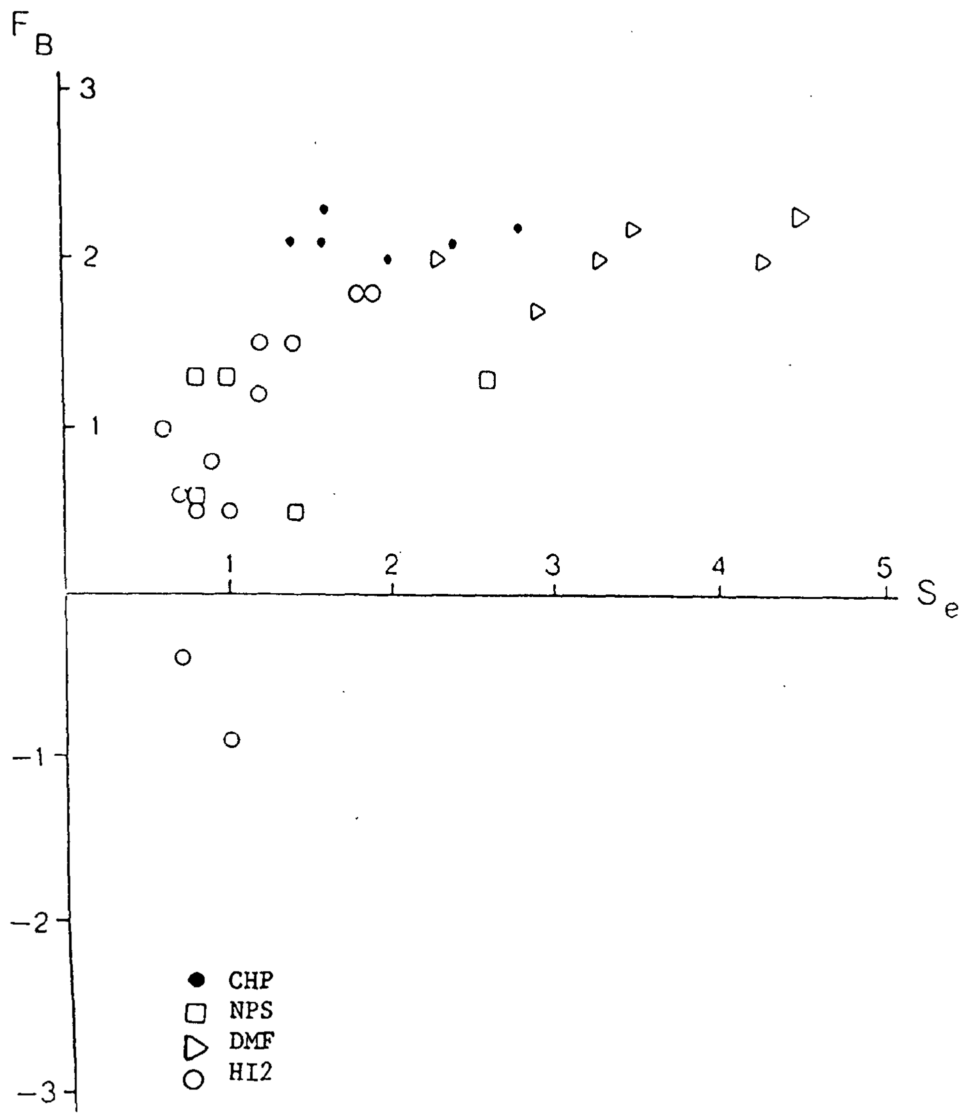


Figure 61. Relation between S_e and F_B.

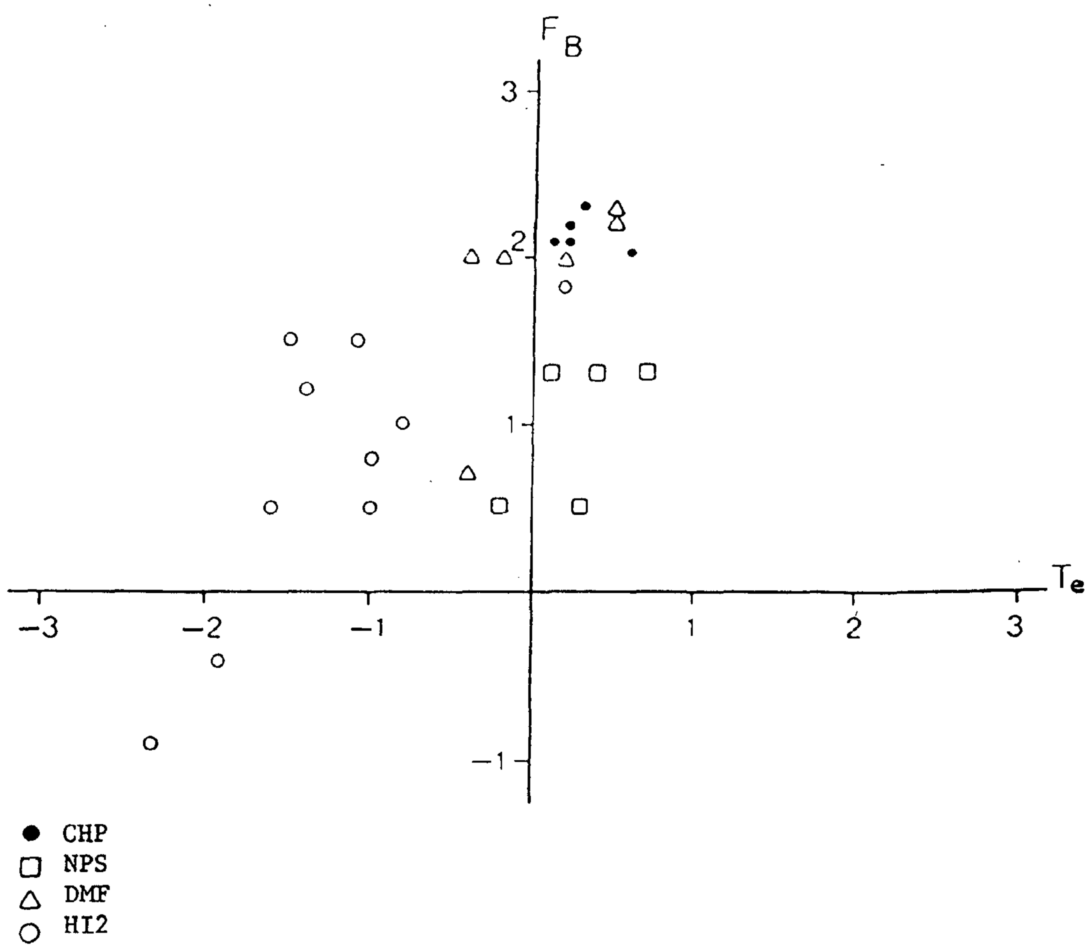


Figure 62. Relation between T_e and F_B.

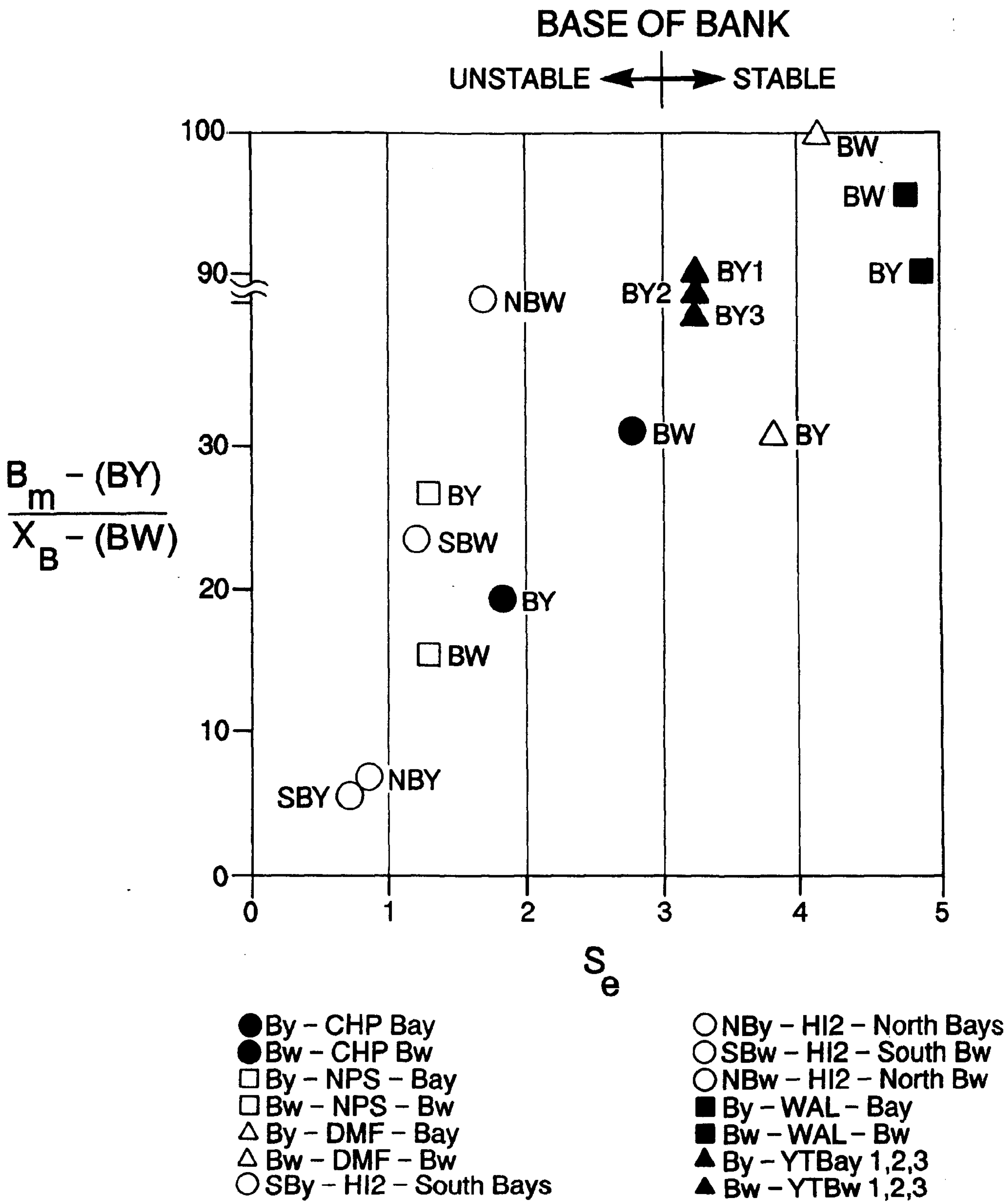


Figure 63. Bank stability relationships - backshore beach elevation vs. (S_e) and (B_m)/(X_B).

Unstable base of banks occurs at Parkway breakwaters, Hog Island breakwaters, and Chippokes. The breakwater units at these sites were placed at or just below MLW. At this time, they do not allow enough distance for the development of a stable backshore beach width, even at Hog Island breakwaters where beach fill was placed. Further bank erosion may create the necessary distance. This is especially true at Parkway breakwaters where erosion of the low bank continues to increase the backshore beach width. Someday this will provide a protective beach and stabilize the low bank.

A plot of bay depth (Mb) against beach width (Bm) shows base of the bank stability at Yorktown Bays, Drummonds Field and Waltrip where Bm is greater than 30 feet and Mb is greater than or equal to 50 feet (Figure 64). The deeper pocket beaches (Mb) are attained when breakwaters are placed over 90 feet offshore, XB (Figure 65) and gaps, Gb, are greater than 75 feet (Figure 66). Large volumes of beach fill, such as at Waltrip, will provide a wide backshore with the bay shorelines being formed with lesser gaps and bay depths.

An analysis of the April 1988 northeaster showed slight beach deflation and little offshore transport of beach sands at the breakwater sites. A change in beach sand grain sizes was also noted as a result of the storm, but no clear correlation to the storm wave climate could be measured. The response of the beach planforms to wave processes in specific storm events will be the focus of next year's final monitoring report.

Headlands

Shore response within the embayments between headlands on open coasts is, in part, a function of incident wave direction (Silvester, 1974). In

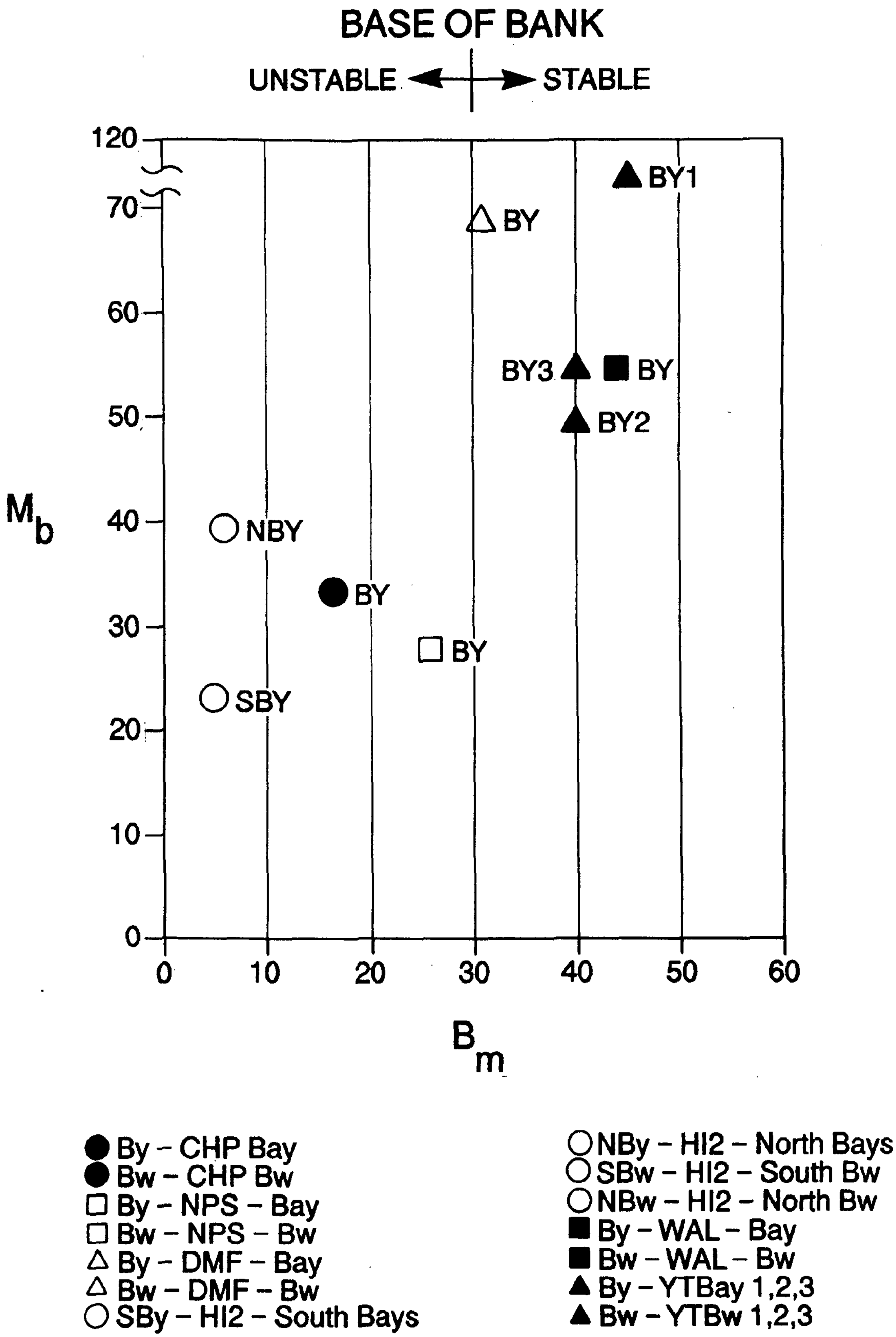
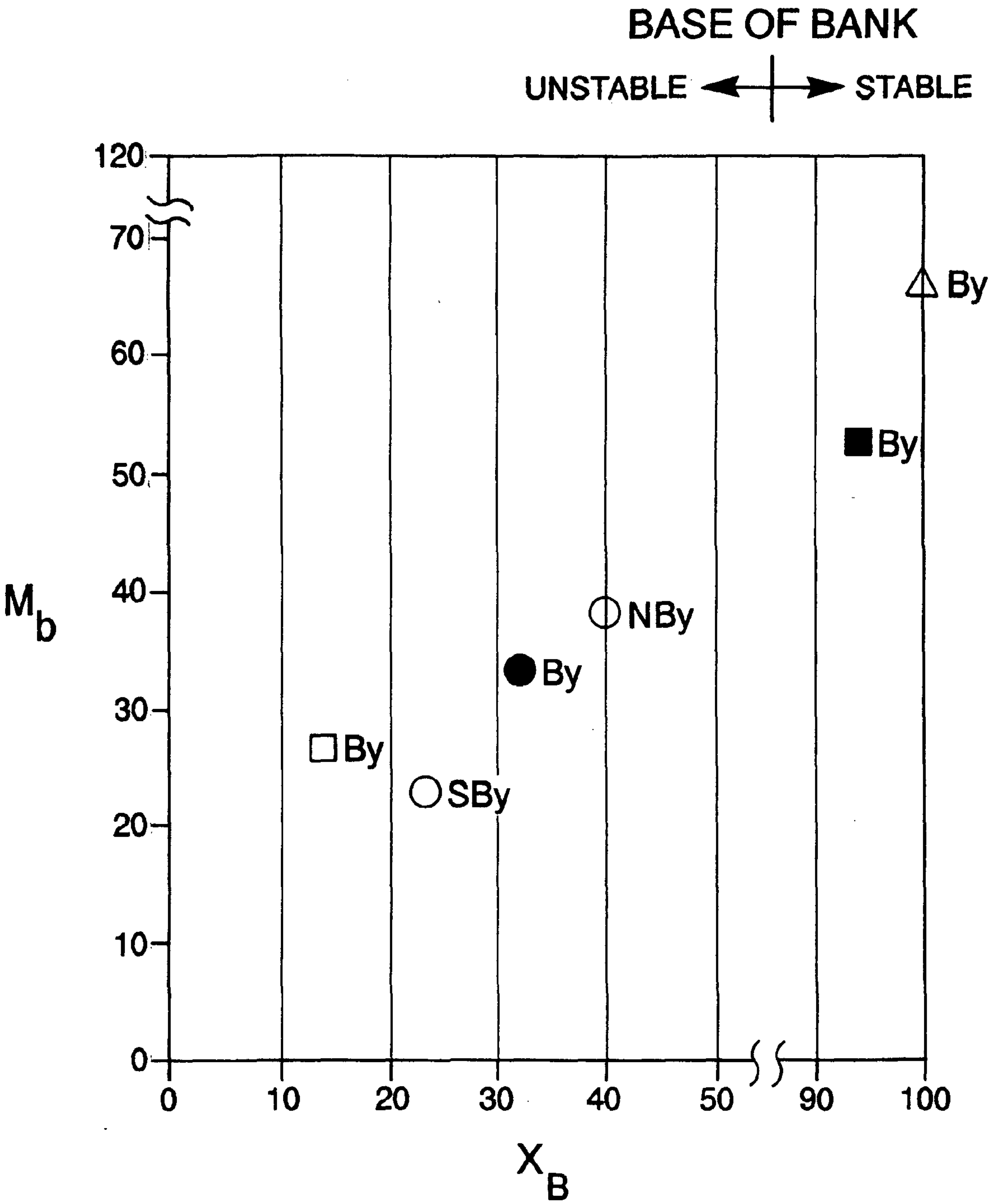


Figure 64. Bank stability relationships - B_m and M_b.



● By – CHP Bay
● Bw – CHP Bw
□ By – NPS – Bay
□ Bw – NPS – Bw
△ By – DMF – Bay
△ Bw – DMF – Bw
○ SBy – HI2 – South Bays
○ NBy – HI2 – North Bays
○ SBw – HI2 – South Bw
○ NBw – HI2 – North Bw
■ By – WAL – Bay
■ Bw – WAL – Bw
▲ By – YTBay 1,2,3
▲ Bw – YTBw 1,2,3

Figure 65. Bank stability relationships - X_B and M_b.

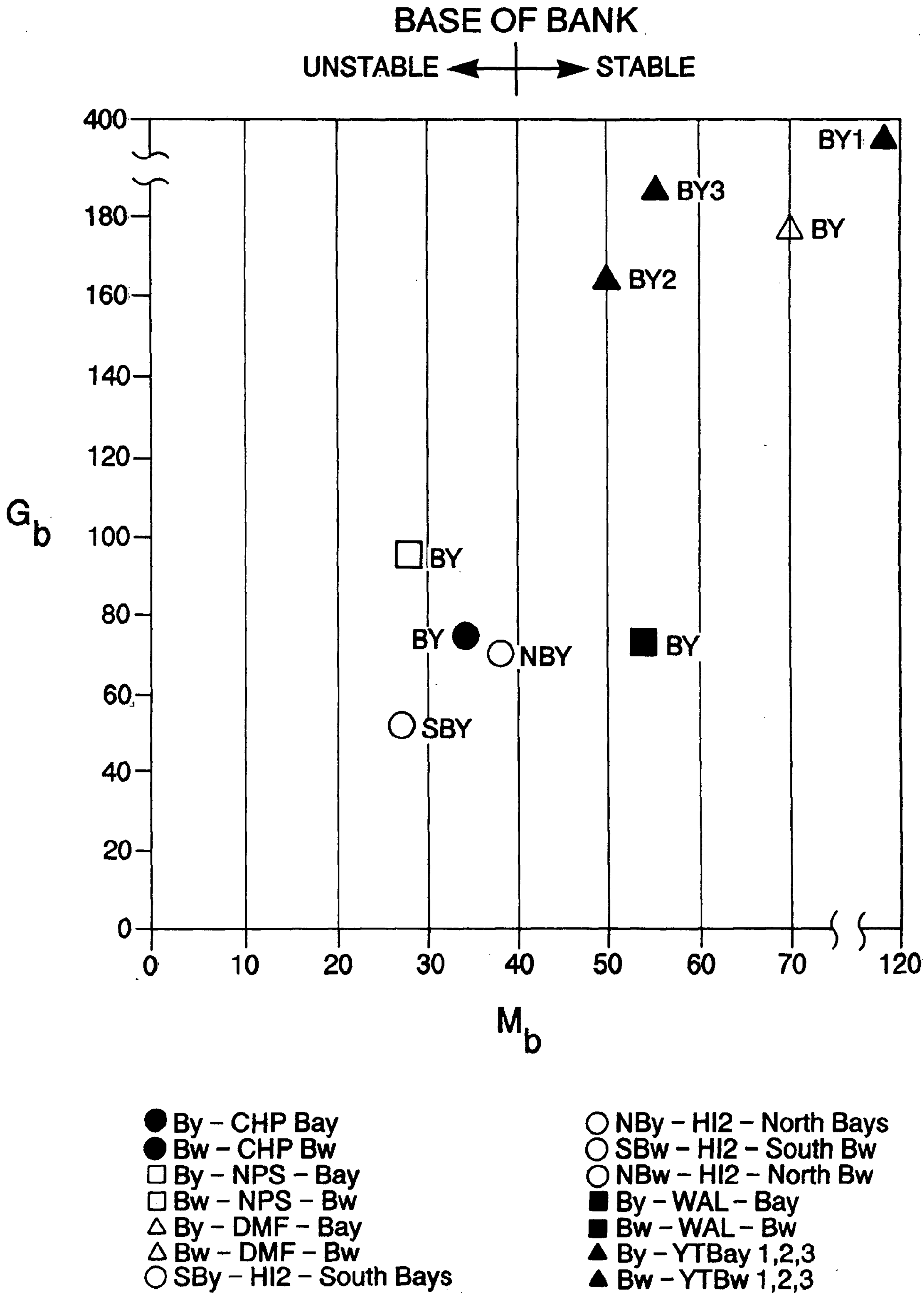


Figure 66. Bank stability relationships - M_b and G_b.

the case of the three designated headland sites of the Chesapeake Bay Shoreline Study, this would be the net incident wave direction because of the seasonality of the wave climate. Last year, we used Silvester's model of stability criteria for equilibrium shaped bays for comparison in the Virginia estuaries (Hardaway et al., 1988). It did not really allow the entire bay shape to be evaluated but instead only supplied the maximum bay indentation distance for bay beach equilibrium. Hsu et al. (1989a, 1989b) used something akin to a modified log-spiral approach and empirical data to come up with another model for analyzing pocket beaches. This model is termed Static Equilibrium Bays (SEB).

For the headland sites, the SEB model utilizes two new parameters, an arc of length R angled theta to the wave crest line, which is assumed parallel to the tangent at the downcoast limit of the beach (Figure 67). The point on the upcoast headland where diffraction takes place is generally easy to define.

The downcoast control point may not be so easy to recognise, especially if a headland protrudes into the bay such as the end of a breakwater (Figure 68). There it is seen that wave diffraction in the shadow zone could cause an almost circular beach form which joins the main bay shape at some transition point. It is the tangent at this point that dictates the orthogonal used for computing the stable bay shape. The length of R then can be computed at given angles from the wave crest (theta) around the bay (Figure 69).

The SEB model was applied to the headland sites on the bay at Summerille, bay B at the Hog Island Headlands and bay 1 at the Yorktown Bays (Figure 70). The direction of wave approach was determined from the orientation of the tangential section of each embayment. The predicted

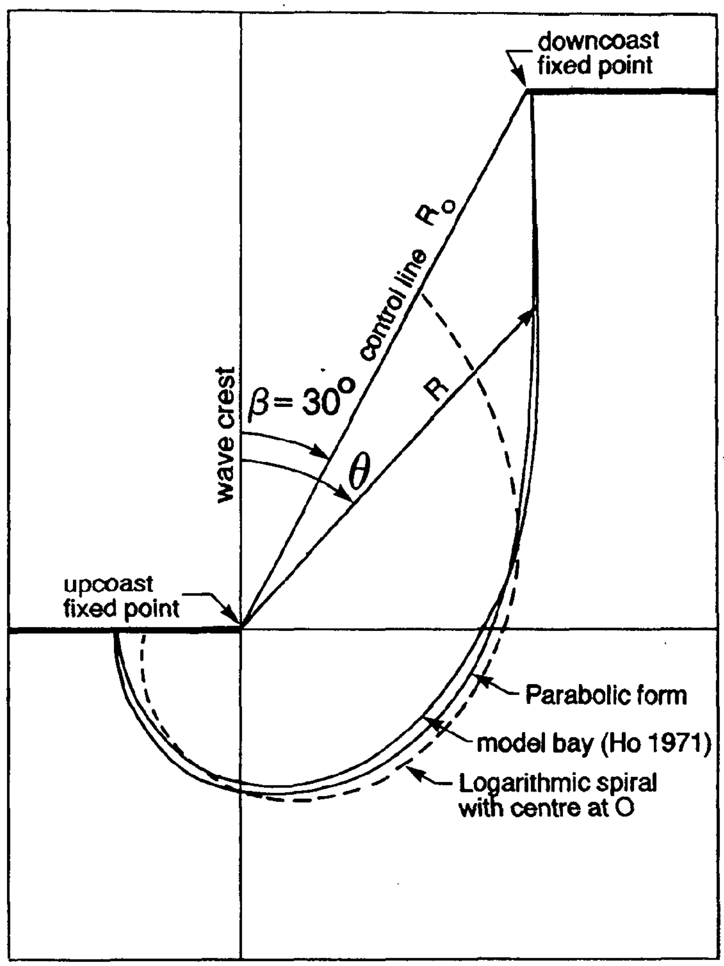


Figure 67. Hsu's static equilibrium bay (SEB) model.

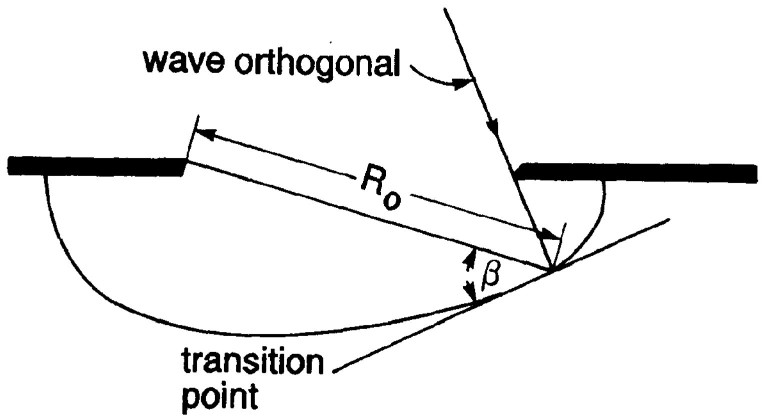


Figure 68. Static equilibrium bay determination of R_o + B (after Hsu et al., 1989a).

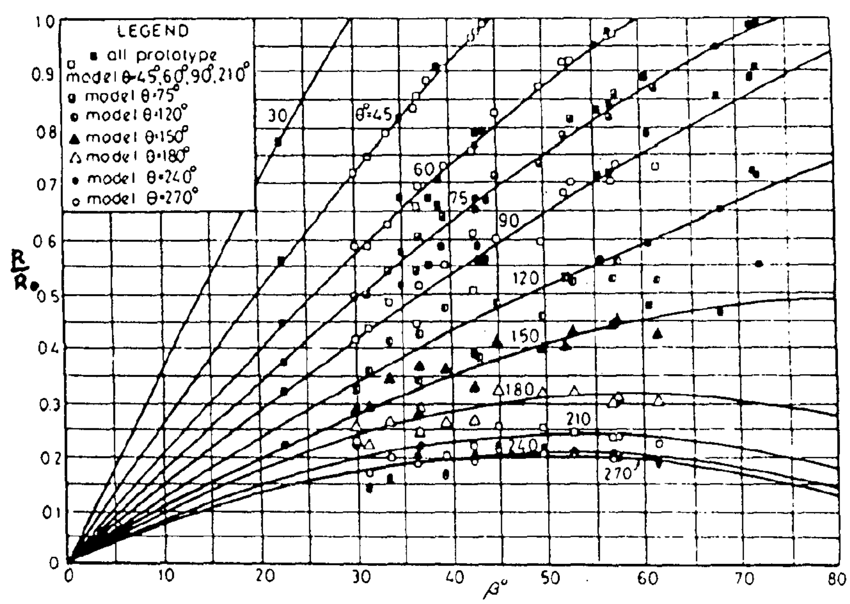


Figure 69. Empirical relationships for static equilibrium bays (after Hsu et al., 1989a).

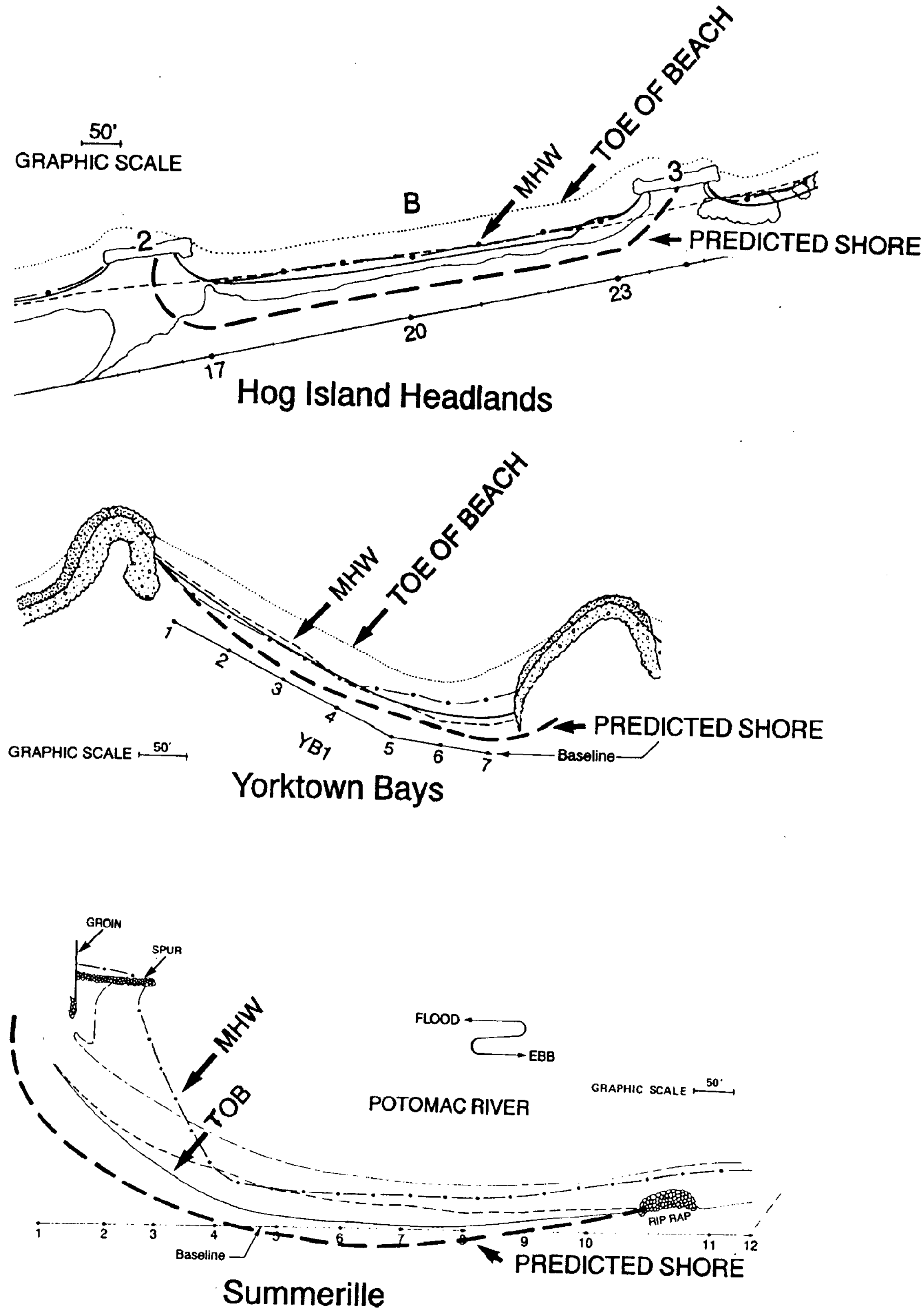


Figure 70. SEB model applied headland sites.

planform for each bay appears to correspond to the top of the bank at Summerille and Hog Island bay B and to about 3.0 feet above MHW at Yorktown Bay 1. The SEB model shows that Hog Island headlands and Yorktown Bay 1 are in near equilibrium. At Summerille, further bank erosion is predicted from profile 3 to profile 6 before equilibrium is reached.

The situation at Summerille is difficult to ascertain due the low diffraction point which is the end of the spur. Under storm surge conditions, the spur is overtopped and the diffraction occurs at the base of the bank. If the predicted planform is MHW, the bank must erode back an additonal 40 to 50 feet in order to provide a protective beach. At the Hog Island headlands the same situation applies where if the predicted planform is along MHW then the bank must continue to erode to achieve equilibrium.

The stability of the base of the bank at the headland sites at this point only occurs at the Yorktown Bays. The fastland banks at Summerille and Hog Island headlands are and will continue to erode in theory to a stable planform.

This model should be considered another tool in the assessment of pocket beaches for shoreline erosion control. The previous log-spiral model of Silvester is still useful as a secondary check for stability (Hsu et al., 1989). Further research into the applicability of the SEB model is needed under using different scenarios of wave angle approach and storm surge. The headland sites and the breakwater sites will be further evaluated by the SEB model and for storm reponse characteristics during the third and final year of the Chesapeake Bay Shoreline Study.

Storm Response

The April 1988 northeaster reduced beach slopes in the bays between the headlands. Also, continued bank erosion was documented at Summerille. Generally, a return of the beach sands was observed. Maximum offshore movement was less than 20 feet at any one beach. This must be taken into account when placing headland units. Transport of beach sands beyond the limit of the headlands may mean the permanent loss of the fill.

Discussion

The definitive protective beach/breakwater system must be designed to withstand given storm conditions including the consequent surge. The Yorktown Bays, although a unique situation, offer a long term, stable series of pocket beaches with exposure to a relatively high wave climate for comparison to other sites. The problem may be the cost required to simulate the same situation on other shorelines.

The test of site success would appear to be the long term stability of the base of the bank. For the breakwater sites, this would include Drummonds Field and Waltrip. Drummonds Field and Waltrip were artificially nourished and both sites have similar fetches. Both sites are protected from the north and northeast wind directions but have a long fetch to the southeast and west. Chippokes, Parkway Breakwaters and Hog Island breakwaters will continue to adjust mainly by fastland erosion.

A site analysis along project reaches must be done pursuant to the installation of widely spaced breakwaters to create a headland/bay situation. Shore morphology evaluation has shown to be valuble in this aspect. The Yorktown Bays are geomorphically isolated, relatively stable, pocket beaches. Summerille evolved into its present configuration since

1967. At Hog Island headlands, the shoreline had evolved into a planform which suggested the feasibility of using headlands for erosion control.

It appears that recent modifications to older shoreline response models involving pocket beaches provide good fit indicators for bay beach equilibrium state. Further process response analysis utilizing wave data and beach surveys are needed to "fine tune" those models for use in the Chesapeake Bay. These models, along with the geomorphic expression of a shoreline, especially the fastland configuration, will provide the long term response to the seasonal and storm induced wave climate. How a given breakwater or headland system withstands storm conditions and provides adequate shore protection will be the final test of success.

Conclusions

At this point in the study some general trends can be stated;

1. For storm surges of +2.0 feet MHW bank protection is provided when the backshore beach width and backshore beach elevation are 30 feet and 3.0 feet respectively.
2. Protective beaches for medium wave energy conditions are best created when beach fill is used in a breakwater project and the breakwaters are placed at least 50 feet offshore.
3. Predicting shore planform can be done using models as developed by Silvester and Hsu in combination with analysis of shore morphology and wave climate.
4. Further work is needed in evaluating wave climates in the Chesapeake Bay especially beach responses to storm conditions.

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